NEW JERSEY DEPARTMENT OF ENVIRONMENTAL PROTECTION
TRENTON, NEW JERSEY

FINAL CONCEPTUAL DESIGN REPORT

REMEDIAL INVESTIGATION/FEASIBILITY STUDY
COMBE FILL SOUTH LANDFILL

June 1987

LMSE-87/0211&455/106

LAWLER, MATUSKY & SKELLY ENGINEERS
as Prime Contractor
in Association with
R.E. WRIGHT ASSOCIATES, INC.

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PREFACE

This report on the conceptual design of the recommended remedial alternative for the Combe Fill South Landfill is the final in a series of three reports on this Superfund site prepared by Lawler, Matusky & Skelly Engineers (LMS) in association with R.E. Wright Associates, Inc. (REWAI). The conceptual design should be reviewed in conjunction with both the remedial investigation (RI) report (LMS 1986a) and the feasibility study (FS) report (LMS 1987) previously submitted. Some information is referenced from these reports rather than repeated here.

CHAPTER 1

DESCRIPTION OF SELECTED REMEDIAL ALTERNATIVE

Based on the evaluation of remedial action alternatives developed as part of the feasibility study (FS) for the Combe Fill South landfill (LMS 1987), a recommended alternative was formulated that consisted of remedial components from several alternatives. The following paragraphs discuss the rationale used in selecting the final recommended remedial components that are summarized in Table 1-1. Figure 1-1 schematically shows the location and relationship of most of the remedial components on a plan view map of the site.

The recommended alternative includes the provision of permanent public water to residents at risk, with interim use of bottled water until the new permanent source is provided. The remedial investigation (RI) has demonstrated, based on hydrogeological investigations and groundwater quality sampling, the off-site movement (primarily northeast and southwest) of contaminants in the drinking water aquifer at concentrations that may be a threat to public health. The alternate water supply study (LMS 1986b) concluded that the most feasible alternate water supply source is the Washington Township Municipal Utilities Authority (WTMUA). Although a final service area has not yet been delineated, at a minimum, public water should be provided to the residents of Schoolhouse Lane and Parker Road from the vicinity of Trout Brook to the intersection of Schoolhouse Lane and Parker Road.

Security fencing with a locking gate will be installed around the perimeter of the remediated fill area and the on-site treatment facility. Although not a deterrent to determined trespassers, the

TABLE 1-1

RECOMMENDED REMEDIAL ALTERNATIVE[a]

Combe Fill South Landfill

1. An alternate water supply for affected residences and security fencing to restrict access
2. Capping of the 65-acre landfill in accordance with Resource Conservation and Recovery Act requirements
3. An active gas collection and treatment system for landfill gases
4. Surface water controls to accommodate seasonal precipitation and storm runoff
5. Pumping and on-site treatment of shallow groundwater and leachate, with discharge to Trout Brook
6. Appropriate environmental monitoring to ensure the effectiveness of the remedial action
7. A supplemental feasibility study to evaluate the need for remediation of the deep aquifer

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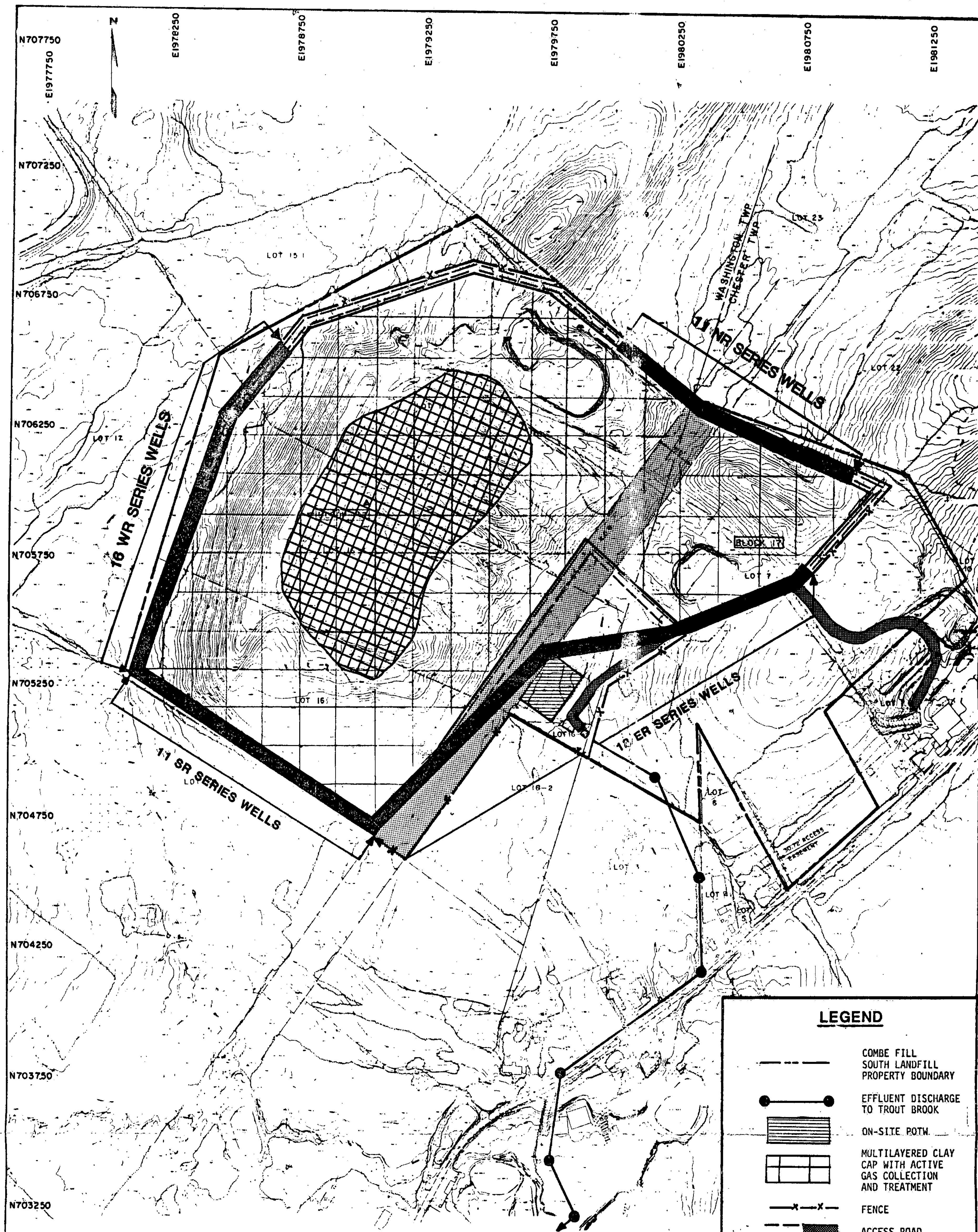
[a]As presented in the record-of-decision (ROD).



LEGEND

- COMBE FILL SOUTH LANDFILL PROPERTY BOUNDARY
- EFFLUENT DISCHARGE TO TROUT BROOK
- ON-SITE POTW
- MULTILAYERED CLAY CAP WITH ACTIVE GAS COLLECTION AND TREATMENT
- FENCE
- ACCESS ROAD
- EXCAVATED WASTES MOVED TO MAIN AREAS UNDER CAP
- SHALLOW WELL PUMPING AND TRANSMISSION SYSTEM
- PARTIAL IMPERMEABLE MEMBRANE

400 0 400

Scale in feet

Lawler, Matusky & Skelly Engineers
Environmental Science & Engineering Consultants
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Pearl River, New York 10965

CONCEPTUAL DESIGN:
COMBE FILL SOUTH LANDFILL
REMEDIAL INVESTIGATION/
FEASIBILITY STUDY

r. e. wright associates, inc.
earth resources consultants
harrisburg pennsylvania

SCHEMATIC PLAN VIEW
RECOMMENDED ALTERNATIVE

FIGURE 1-1

fence will prevent most direct physical contact with the general public.

Grading, filling, and general site preparation and the installation of a paved access road to the on-site treatment facility and a dirt access road around the fill perimeter are the necessary precursors to the construction of the multilayered cap.

A 72-acre multilayered, terraced cap with a clay layer having a permeability of 10^{-7} cm/sec will be constructed over the regraded fill surfaces, including the areas under the power line right-of-way. This capped area consists of:

- Approximately 65 acres of land actively used for landfill operations (although the Combe Fill Corporation owns a total of 115 acres at the site)
- Approximately 7 acres underneath the power line right-of-way and land next to active fill areas, which will be capped to provide continuity and smooth edges for the single cap

Where technically possible, the multilayered cap will include an impermeable membrane between the sand drainage layer and the clay. This membrane will cover about 16 acres of the fill area and is included in this remedial component because its use is consistent with established EPA policy to comply fully with Resource Conservation and Recovery Act (RCRA) in remediating Comprehensive Environmental Response and Liability Act (CERCLA) sites.

Active gas venting and treatment, consisting of a network of 65 gas extraction wells connected to vacuum blowers, will provide positive control over landfill gases. Proposed treatment technologies include flaring for removal of methane and some volatile organics.

Permanent surface water controls, including berms, reinforced drainage chutes, gabion terraces, and a circumferential drainage ditch, will be needed to direct and control stormwater runoff at the remediated site. Temporary surface water control measures must also be employed during construction at the site.

Contaminated groundwater/leachate will be controlled with a perimeter shallow aquifer pumping system consisting of extraction wells tapping the saprolite aquifer. This system was considered to be less expensive, technically more feasible, and probably more effective than a leachate trench in controlling the movement of contaminated groundwater in the saprolite aquifer, where most of the groundwater flow occurs. Questions concerning the effectiveness, demonstrated technical need, and potential impacts of a bedrock pumping system should be addressed in a supplemental feasibility study to evaluate the need for remediation of the deep aquifer and the need to further dewater the capped waste.

The shallow aquifer pumping system will send contaminated groundwater to an on-site treatment facility for complete physiochemical/biological treatment prior to discharge to Trout Brook. On-site treatment with discharge to Trout Brook is more cost-effective than either off-site final treatment with on-site pretreatment or treatment with discharge to Black River.

An expanded environmental monitoring program that provides for extensive monitoring of the shallow and deep groundwaters on- and off-site should be undertaken both during and after construction of the remedial components. Such monitoring information is necessary to further define the extent, speed, and direction of contaminant movement off-site so that decisions can be made about the need for additional remediation, e.g., deep aquifer pumping or further extension of public water.

The design criteria, performance expectations, and operation and maintenance needs of the technical components of the recommended alternative are further defined in Chapters 2 and 3 of this report. The proposed long-term environmental monitoring program is detailed in Chapter 4, and the proposed treatability study is described in Chapter 5. Chapter 6 discusses the design and implementation problems, defines areas of additional data needs, and identifies possible permit requirements of the recommended alternative. Cost estimates for the recommended alternative are summarized in Chapter 7, and a design/implementation schedule is discussed in Chapter 8.

CHAPTER 2

PRELIMINARY DESIGN CRITERIA AND PERFORMANCE EXPECTATIONS

2.1 SITE ACCESS AND SECURITY

Access to the site and treatment facility will be afforded by an 18-ft wide paved road with a total length of 2200 ft extending from Parker Road to the on-site treatment plant. A paved road will be necessary to accommodate the truck traffic to and from the on-site treatment plant. The road, as proposed and shown in Figure 1-1, requires either an easement or a right-of-way through adjacent private property from the site boundary to Parker Road. Approximately 7300 ft of an unpaved gravel road will circle around and provide access to the cap and its appurtenances. Access to the terraced areas of the cap will be along the western face of the cap using an all-terrain vehicle. Plate 1 provides a plan view of the terrace access routes.

A 6-ft high chain-link fence, approximately 8000 ft long (Plate 1), will encircle the remediated site areas and the treatment facility. A locking gate will be located at approximately the junction of the eastern property border and the new paved access road. Signs warning that the site is a remediated hazardous waste site will be posted every 50 ft (or as often as required, depending on line-of-sight) around the site and in specific working areas. These security measures will discourage most of the general public, thereby reducing their risk of exposure, but will not stop the determined trespasser or vandal.

2.2 GRADING, CAPPING, AND TERRACING

Prior to capping, the existing wastes must be partly excavated and regraded to minimize the amount of fill to be capped and to provide appropriate base and slope conditions for the cap layers. Approximately 210,000 yd^3 of waste/soils will be excavated and regraded on-site.

Wastes within the right-of-way of the New Jersey Power and Light Company (NJPLC) power line will be excavated to at least 6 ft so that the cap placed underneath the power line will be no higher than the current land surfaces. The wastes excavated from the right-of-way will be graded into the major waste piles located east and west of the right-of-way. It has been assumed for costing purposes that all waste excavation and regrading will require, at a minimum, Level C personnel protection equipment and may often require Level B protection. Despite the high costs associated with work under such health and safety conditions, regrading of waste is less expensive than filling and regrading with purchased local borrow.

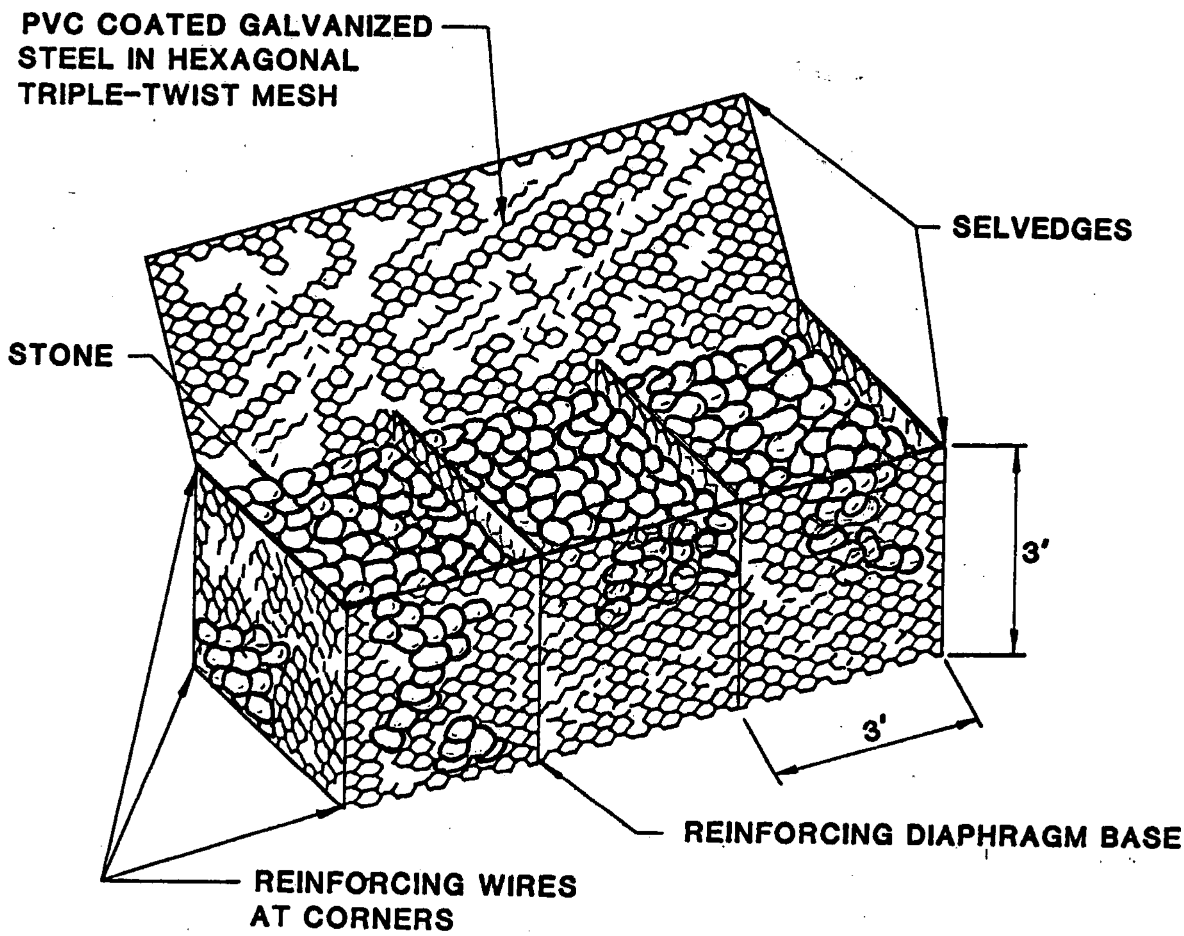
After the waste has been regraded about 60 acres of the waste area will have recontoured slopes less than 20% (the slope above which erosion and slippage may seriously undermine the cap) and will be suitable for the placement of the cap layers. Plate 1 shows the approximate final regraded landfill contours before capping.

The regraded, but still too steep, remaining 12 acres of waste along the western edge of the landfill will require the construction of six tiers of gabion terraces to control runoff and prevent serious erosion. Figure 2-1 shows the details of the PVC-coated, galvanized steel mesh cages that are filled with 4 to 8-in. diameter stones to make up the gabion sections. These sections are

FIGURE 2-1

GABIONS

Combe Fill South Landfill



NOT TO SCALE

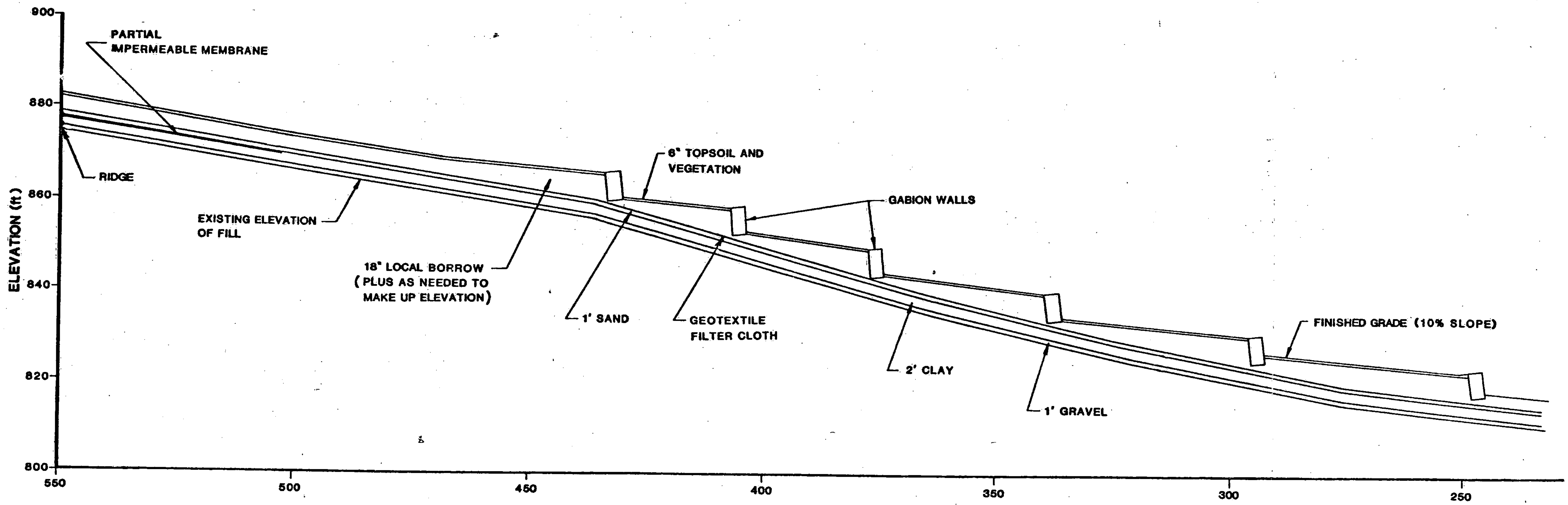
placed and secured together to form walls that are anchored into the second (from the top) layer of the cap (i.e., the 18-in. layer of local borrow beneath the topsoil and above the geotextile fabric). Additional amounts of borrow are used to fill in behind each gabion wall. Figure 2-2 shows the cross-sectional relationship of the gabion terraces and cap layers along the western face of the cap. The possibility of structural failure or collapse of the two-tier-high gabion wall terraces used in the cap is very small. The wall's structural capacity is well in excess of the expected loadings. The inherent flexibility of the rock-caged gabions would allow the wall to deflect rather than fail as the result of differential settling of the supporting soils under the gabion walls. Gabions are designed for atmospheric and submerged exposures and are generally resistant to corrosion. Normal life exceeds 20 years in submerged exposures, with longer life in atmospheric exposures. If corrosion does take place and wires of the mesh fail, the general effect will be to loosen the stones in the gabion cage. Should cages corrode and remain unrepaired, the result would be a rubble stone wall, tapering toward the top, which would change the shape of the exposed face of the cap terrace; however, collapse of wall sections is unlikely even under earth pressures. If a section of wall were to collapse, the terrace upstream of the gabion would erode and gullying would result. Since gabion walls would be installed above the clay layer of the cap, long-term erosion would be required before the cap itself failed. It is assumed that maintenance procedures would be undertaken well in advance of such an occurrence.

The multilayered clay cap, with partial synthetic membrane (see Figure 2-3), is designed to achieve several remedial objectives, including:

FIGURE 2-2

CROSS SECTION OF WEST FACE OF LANDFILL, CAP AND GABION TERRACES[a]

SECTION A-A



DISTANCE FROM CENTERLINE OF WEST BRANCH TROUT BROOK (FT)

0-00 = ℄ TROUT BROOK

[a] GABIONS @ APPROX. LOCATIONS SHOWN ON PLAN OF FINAL GRADING, PLATE 1

represent an average reduction in internal landfill water levels (saprolite aquifer) of 15 to 20 ft. This would mean that at several locations in the fill the projected lowered water levels would be above the estimated bottom of the waste pile, i.e., groundwater will still intersect waste piles in some areas of the fill. However, in conjunction with the recovery well system described in Section 2.5, this equilibrium water table will experience drawdown at the landfill perimeter such that 95% of the waste will lie above the groundwater table, as shown in Figure 2-4. Section 2.5 of this chapter provides additional detail on the shallow pumping well system and its effects on leachate discharge from the capped landfill.

2.3 SURFACE WATER CONTROLS

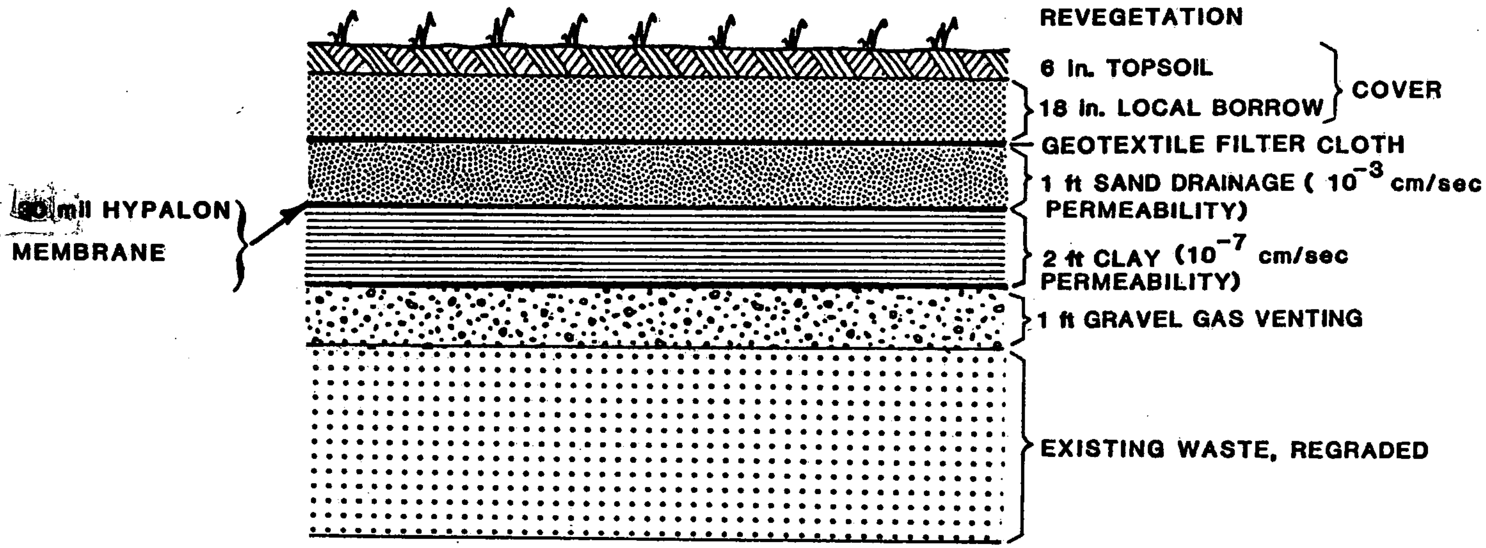
In addition to the cap surface revegetation, contouring, and terracing previously described, the structural elements of the surface water control systems for the site will consist of berms, reinforced chutes, and a paved perimeter drainage ditch (see Plate 1). Two tiers of earthen berms will be used to control and direct surface runoff along the southeastern face of the cap to a drainage chute leading to the site perimeter drainage ditch. Higher velocity runoff flows along the western face of the cap are directed by cap contours and terracing to a reinforced drainage chute (i.e., horizontal gabions) that also discharges to the circumferential drainage ditch. The paved perimeter drainage ditch, located between the access road and the security fence, encircles the entire cap. At several points along the ditch outlets are provided to the headwaters of the East and West branches of Trout Brook.

Assuming complete impermeability of the 72-acre cap and no additional losses of water due to increased plant uptake or evapotranspiration from the recontoured and revegetated site, the total run-

FIGURE 2-3

CLAY CAP WITH IMPERMEABLE MEMBRANE SECTION VIEW[a]

Combe Fill South Landfill



- Direct remediation of the air contaminant pathway by controlling the emission of volatile organics, landfill-generated methane, and contaminated dusts into the air

- Direct and indirect remediation of the surface water contaminant pathway by preventing the direct contact of rainfall with the wastes, thereby minimizing leachate production

- Indirect remediation of the groundwater contaminant pathway by minimizing rainfall infiltration, thereby substantially reducing the production of leachate that contaminates the groundwater.

The 6-ft cap will consist of:

- 1 ft of gravel, placed on the regraded waste and used as part of the gas venting system

- 2 ft of clay, graded and compacted to achieve a permeability of 1×10^{-7} cm/sec or less

- A 30-mil Hypalon membrane in portions of the site

- 1 ft of sand (permeability 1×10^{-3} cm/sec) to be used as a drainage layer, connected at the cap perimeter to the surface water runoff system

- A geotextile filter fabric placed above the drainage layer to prevent clogging of the sand layer from fines percolating through the top cap layers

- 18 in. of local borrow and 6 in. of top soil as a final cover, which will be revegetated with grasses to help prevent surface soil erosion

As shown in Figure 1-1, a synthetic impermeable membrane can be used in approximately 16 acres of the western half of the landfill where regraded slopes are less than 7%. Placement of a membrane in areas where slopes are greater than this may result in slippage of the upper layers of the cap.

Because use of a synthetic membrane as one of the cap layers is in keeping with RCRA cap design recommendations, a 30-mil Hypalon membrane has been included in the portion of the cap that has suitable slopes: the 16 acres in the western half of the site. The effectiveness of this membrane for further reducing infiltration was evaluated in a scenario in which the clay layer is completely saturated. In such a scenario the total infiltration flow through the cap is calculated to be approximately 4.75 gpm over the capped area of the landfill. This is about 7000 gpd, or about 7% of the total average infiltration through the cap. Assuming that the membrane is completely impermeable over its 16 acres, the net estimated maximum leakage through the cap under these same conditions would be 3.7 gpm. This is equivalent to about 5200 gpd, or an additional reduction of about 1800 gpd of leakage through the cap because of the membrane. Without the 16-acre membrane the cap would prevent about 93% of the current infiltration; a cap with the membrane would prevent about 95% of the total current infiltration.

Assuming an initial average rate of discharge for the saprolite aquifer of about 100,000 gpd, an initial transmissivity of 1187 gpd/ft, and an initial average hydraulic gradient of 0.0167 ft/ft, Darcy's Law was used in a mass balance calculation to determine the post-capping decline in groundwater/leachate levels (LMS 1986a) in the absence of groundwater pumping (Appendix A discusses the hydraulic properties of the saprolite and bedrock as determined during the RI Study [LMS 1986a]). The decreasing hydraulic gradient that occurs with time under the landfill results in a diminishing rate of reduction of groundwater/leachate such that almost 12 years are required to achieve an 85% reduction of the groundwater discharge rate and saprolite head in the landfill. At such time the landfill water level would reach a relatively stable equilibrium, although an additional 15% reduction in groundwater discharge could occur for the next 20 or more years. This lowered water level will

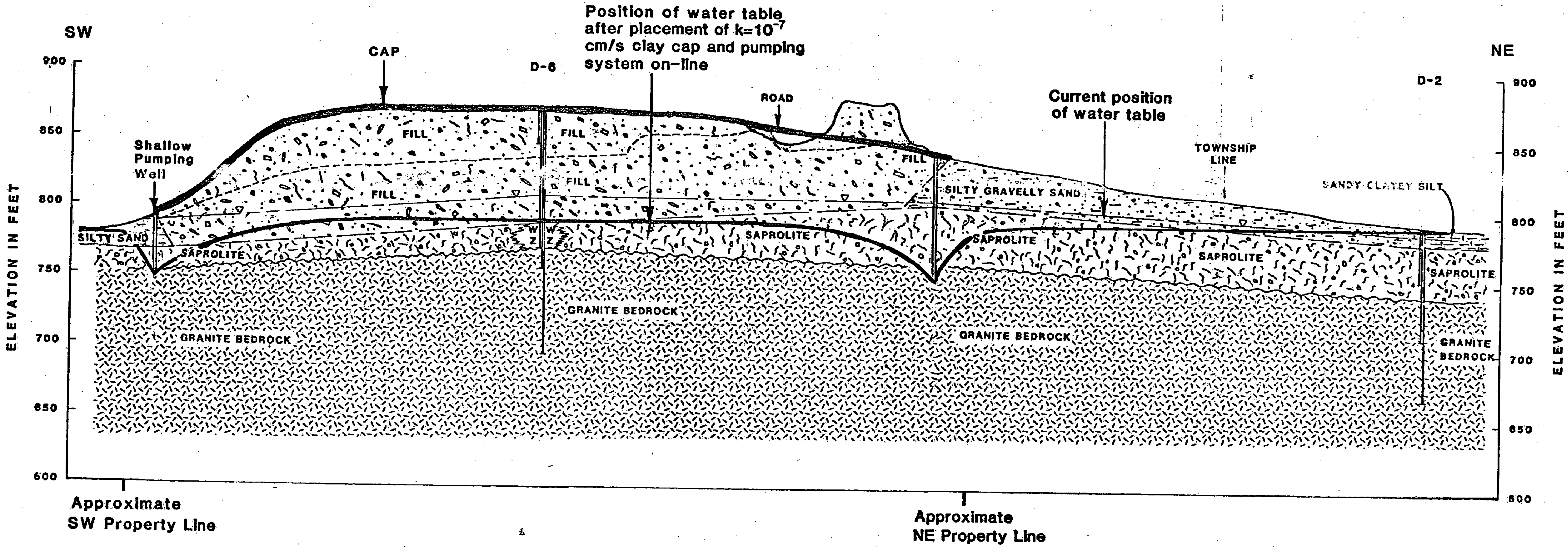


FIGURE 2-4

LEGEND

—— PRESENT GROUND SURFACE

- - - - ORIGINAL GROUND SURFACE

—▽— WATER TABLE

200 100 0 200

HORIZONTAL SCALE

COMBE FILL
SOUTH LANDFILL

WATER TABLE AT
REMEDIATED SITE

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Environmental Science & Engineering Consultants
Pearl River, New York 10965

drawn DLB	approved	drawing no.
checked	date	

r. e. wright associates, inc.
earth resources consultants
middletown pennsylvania

off would be equivalent to the current total ground and surface water discharge.

Assuming that the average annual surface water discharge recorded at the Pottersville gaging station downstream of Trout Brook on the Black River is representative of the landfill site, an approximation of the total annual discharge from the cap can be calculated. The average annual discharge at the Pottersville station is 55.9 cfs or 23.14 in./yr, which is about 46% of the average annual rainfall of 50 in./yr and is the total of direct surface water runoff plus groundwater discharge. Applying this rate to the 72-acre cap, the total discharge from the cap would be 45 x 10^6 gal/yr (124,000 gpd) and would be essentially all surface water runoff. This new total surface runoff is four times greater than the current surface water runoff component, which is estimated to be 25% of the total discharge (i.e., about 31,000 gpd for 72 acres). Most of this runoff will be discharged directly to either the West or East Branch of Trout Brook. These stream segments will therefore have generally higher storm runoff flows than at present. Although not intrinsically necessary for proper functioning of the runoff control system, and not currently included in the recommended alternative, these larger storm runoff flows could be routed to regulated-release detention basins to provide additional control of these flows. At stream discharge points of the runoff control system, the stream channels may have to be widened and their banks reinforced to accept these larger runoff flows. Assuming proper revegetation and maintenance of the cap, sedimentation basins should not be necessary as part of the runoff control mechanisms.

2.4 GAS COLLECTION AND TREATMENT

The active gas collection system of this remedial action consists of a network of 62 gas extraction wells connected by a flexible PVC piping system and common collection headers to vacuum blower facilities near the leachate treatment facility (Figure 2-5). In this manner gases will be actively channeled to a centralized location where flaring will be used to burn off landfill-generated methane and some volatile organics.

The gas extraction wells will be constructed of perforated 4-in. PVC pipe sections installed to the base of the waste pile. The flexible, interconnecting PVC piping system can be constructed within the gravel vent layer or, if easier access and maintenance is desired, within the top two cap layers. Flexible piping is recommended to help prevent breaking or dislodging of the pipe sections as a result of differential settling of the cap layers.

If gas vents are extended above the gravel layer, they will penetrate the clay layer and, in some areas, the impermeable membrane, thus providing opportunities for rainfall infiltration. Tamping and compacting the clay around the vertical gas piping and using heat sealing or adhesives to join the membrane to the vents will help minimize the possibilities for infiltration, but will provide no assurance of long-term membrane/clay integrity or impermeability.

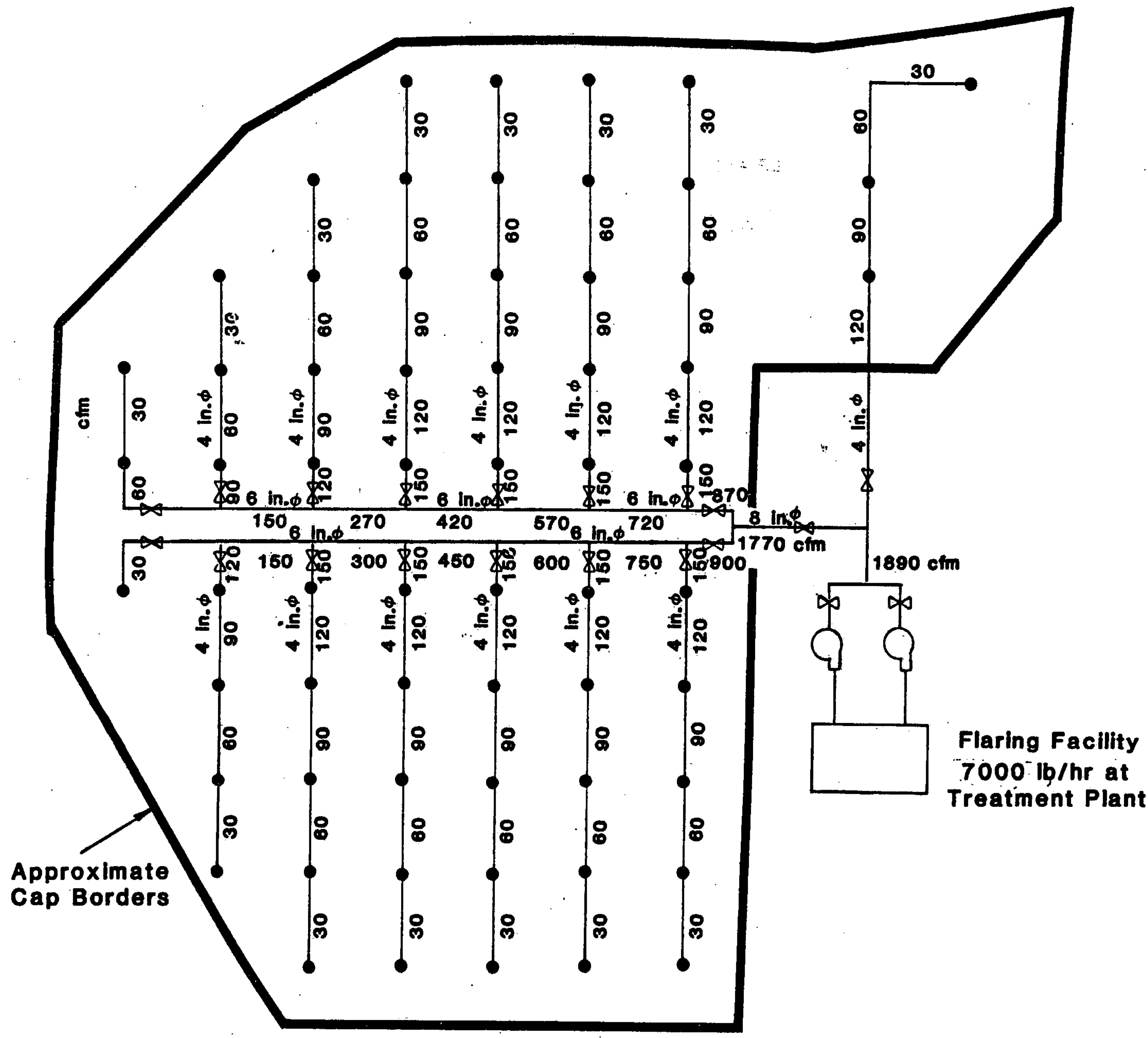
Because only minor amounts of volatile organics were measured during the RI in the ambient air at the landfill, only methane flaring is provided as a gas treatment process. Flaring will oxidize the methane and some minor amounts of volatile organics. If additional air quality sampling reveals that quantities of refractory volatile organics are generated, additional treatment with perhaps carbon adsorption may be necessary.

An active gas collection system has several advantages over a passive system:

FIGURE 2-5

ACTIVE GAS COLLECTION AND TREATMENT [a]

Combe Fill South Landfill



[a]Not to Scale

- It is more effective in collecting landfill gases and is less sensitive to environmental factors that control gas generation and movement.

- Because of its greater effectiveness in removing methane and other gases, it provides greater personnel safety and protection against fire and explosion.

- It allows for the recovery/reuse of methane gas if production of methane is determined to be cost-effective.

Disadvantages of an active gas collection and treatment system are associated primarily with higher operation and maintenance costs, and services and personnel requirements.

Temporary, active gas control will also be necessary during work on the fill areas and during construction of the cap in order to ensure worker safety. These temporary controls may be used subsequently as part of the permanent gas collection and treatment system.

2.5 SHALLOW GROUNDWATER/LEACHATE PUMPING

The objectives of the shallow (saprolite) groundwater/leachate pumping system are to isolate the bulk of the landfilled waste from the groundwater by lowering the water table beneath the fill where possible and to collect shallow groundwater discharge from the landfill for on-site treatment.

To accomplish these objectives the shallow groundwater pumping system will consist of approximately 56 perimeter recovery wells constructed to the base of the saprolite. The wells will pump the shallow groundwater through a network of flexible piping to the on-site treatment facility. The trench carrying the PVC piping will also contain all necessary appurtenances, including the electrical system connecting the wells to the main control center at the treatment facility. Figure 2-6 (two pages) shows the construction details of a typical shallow recovery well. The wells will be spaced about 100 ft on center along the northeast, southeast, southwest, and western perimeters of the waste pile, as shown in Plate 2. They will control 90% of the contaminated groundwater, principally the groundwater in the saprolite at the site.

2.5.1 Well Number and Location

The number and spacing of the shallow recovery well was determined using the Theis method (Freeze and Cherry 1979) relationship of:

$$W(u) = \frac{4\pi Ts}{Q} \text{ and } r^2 = \frac{4uTt}{S}$$

where:

W(u) = well function

T = transmissivity, assumed to be approximately 1187 gpd/ft (LMS 1986a)

s = drawdown at radius r, assumed to be 1 ft to assure overlap of pumping influence

Q = pumping rate, initially assumed to be 2160 gpd (1.5 gpm), based on field observation of effective well yields on-site

r = radial distance from the pumping well in feet

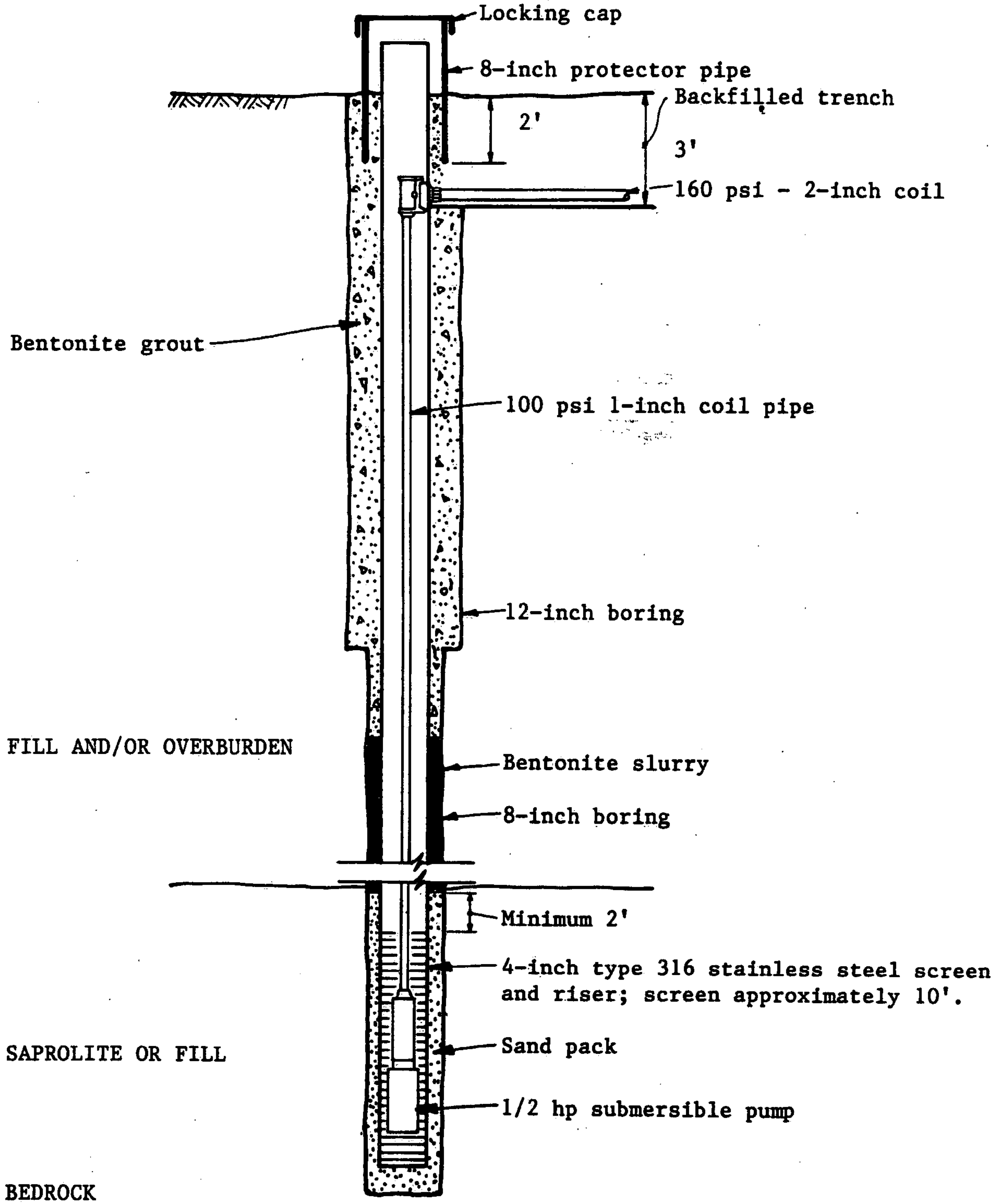
t = time to drawdown s, assumed to be 30 days, based on hydraulic testing on-site

S = specific yield or aquifer storativity, a dimensionless function assumed to be 1×10^{-3}, based on work conducted during the RI (LMS 1986a)

Substituting these parameters in the Theis equations, W(u) and the effective radius were obtained as follows:

FIGURE 2-6

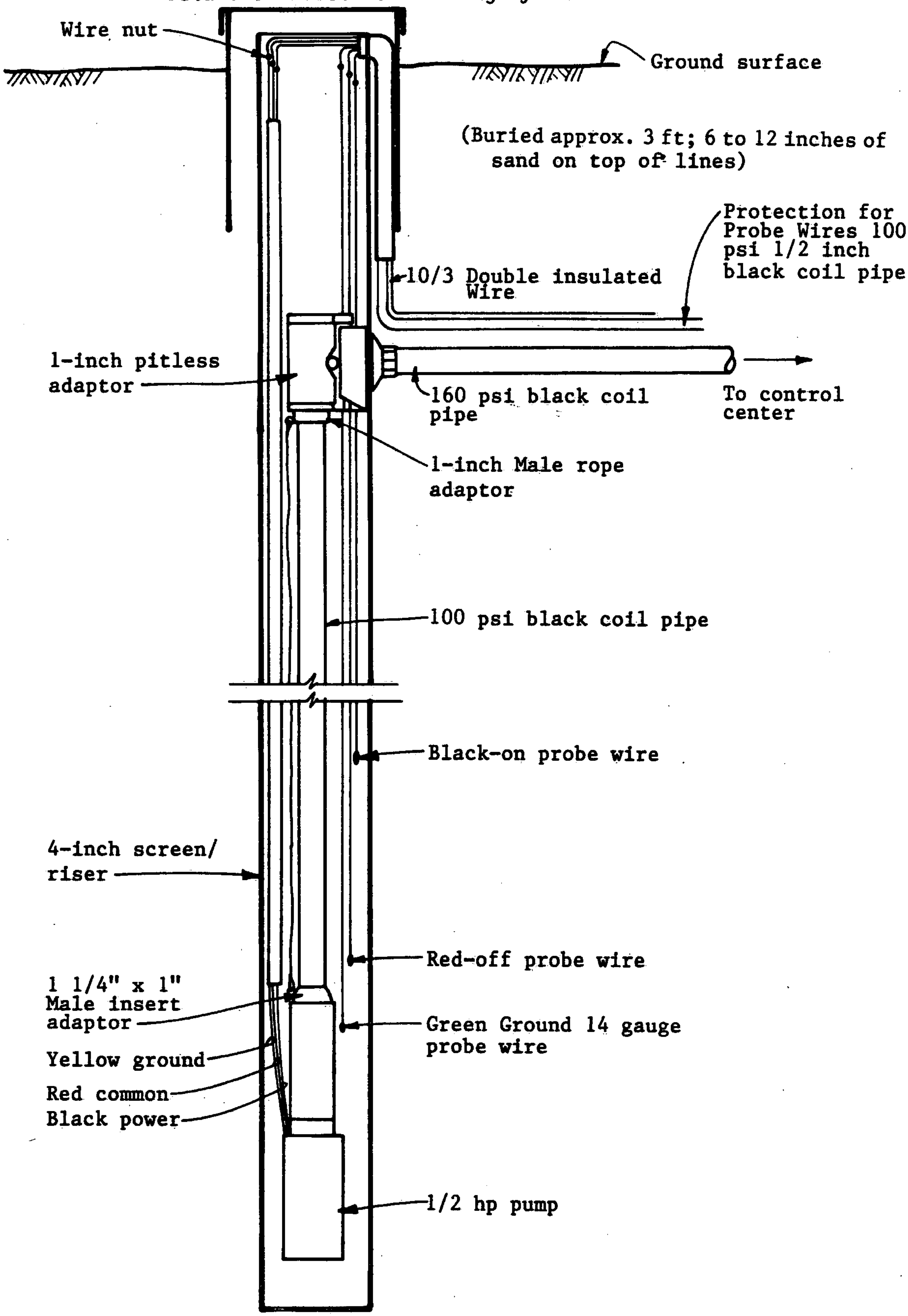
TYPICAL SHALLOW RECOVERY WELL
General Structural Detail



Note: 12-inch outer boring recommended to allow for ease of placement of temporary 8-inch diameter casing sleeve or grouting of permanent casing if required.

FIGURE 2-6 (Cont'd)

TYPICAL SHALLOW RECOVERY WELL
Detail of Electrical Wiring System



$$W(u) = \frac{4 \ (1187 \text{ gpd/ft}) \ (1 \text{ ft})}{1.5 \text{ gpm} \ (1440 \text{ min/d})}$$

$$= 6.90, \text{ so that } u = 5.65 \times 10^{-4}$$

and

$$r^2 = \frac{4 \ (5.65 \times 10^{-4}) \ (1187 \text{ gpd/ft}) \ (30\text{d})}{(1 \times 10^3) \ (7.48 \text{ gal/ft}^3)}$$

$$= 10{,}759 \text{ ft}^2$$

therefore

$$r = 103.73 \text{ ft}$$

Thus, after 30 days of pumping at a rate of 1.5 gpm, an individual well drawdown of 1 or 2 ft overlapping drawdown would be observed within approximately 100 ft of any pumping well. Therefore, a design well offset radius of 50 ft was selected to provide sufficient overlap of the well cones of influence, yet limit the impacts of this drawdown in off-site areas. This radius is equivalent to a well spacing of 100 ft from well centers.

The 100-ft well spacing determined by these methodologies was then applied to each of the six flow channels to determine the total (56) number of wells needed at the perimeter of the site (see Plate 2):

- Eleven recovery wells along the northeastern (NR-Series) perimeter of the site
- Eighteen recovery wells along the southeastern (ER-Series) perimeter of the site
- Eleven recovery wells along the southwestern (SR-Series) perimeter of the site
- Sixteen recovery wells along the western (WR-Series) perimeter of the landfill

Although maximum efficiency of each well would be improved if well alignment were perpendicular to the groundwater flow direction, such an array, besides allowing additional contaminant migration within off-site areas, would also result in a reduction of the proportion of collected contaminated water vs uncontaminated flow from adjacent areas. Furthermore, the flow channels projected in the RI report are not exact and were somewhat arbitrarily defined for the purpose of groundwater flow estimation. The recovery well layout, as conceptually designed, provides the best alternative for restricting off-site migration of contaminants and collection of contaminated groundwater near the source.

2.5.2 Pumping Rates and Impacts

The pumping rates of the shallow well recovery system were determined by two methodologies as described below. Each method assumes that the saprolite generally retains the structure of the parent bedrock, which is anisotropic, and that use of the Theis method (Freeze and Cherry 1979) to determine aquifer transmissivities is appropriate. Both of these conclusions were reached during the RI investigation.

Although it is acknowledged that the Theis equation applies to confined aquifers, use of this methodology to determine aquifer transmissivity for conceptual design purposes is supported by Kruseman and DeRidder (1983) who state: "In an unconfined aquifer in which no effects of delayed yield are apparent, the flow pattern to a pumped well is identified with the flow pattern to a pumped well in a confined aquifer. Consequently,... (the Theis equation) can be used for the analysis of a pumping test in an unconfined aquifer, satisfying certain assumptions." These additional assumptions involve aquifer extent, homogeneity, flat water table, constant discharge rate, and full aquifer penetration. These conditions are reasonably well met on-site to allow for the use of the Theis method (LMS 1986a) for this conceptual report.

The assumption of anistropy for the saprolite aquifer, while reached during the RI (LMS 1986a), is still under question by certain reviewers. Additional data supporting this assumption and the use and appropriateness of the Theis equation for calculating well spacing and pumping rates should be collected and evaluated during the design phase of the study. As discussed in Chapter 6 of this report, long-term pumping tests are needed in order to make appropriate decisions regarding final design of the perimeter shallow well recovery system.

2.5.2.1 Initial Pumping Rate. The initial pumping rate ranging from about 116,000 to 121,000 gpd, was calculated using Darcy's Law and the Theis equation as follows:

Darcy's Law Recharge

Upgradient Recharge + Landfill Recharge + Pumping Rate in Outside Capture Area = Total Groundwater Flow to Be Pumped.

The upgradient recharge to the landfill was determined by applying the calculated annual recharge rate for the landfill area (800,000 gpd/mc^2 or 1250 gpd/acre) to the 5-acres upgradient recharge area, resulting in a total upgradient recharge of 6250 gpd. Similarly, the landfill recharge was calculated as approximately 90,000 gpd based on 72 acres. Finally, the recharge to the pumping capture area outside the landfill was calculated using a variation of the Theis equation presented previously. Instead of a drawdown, "s", of 1 ft, a drawdown of 0.5 ft was used to determine the maximum off-site effects of pumping. Using this revised drawdown, a new radius of drawdown was calculated and was plotted to delineate the downgradient capture area of the pumping wells. This area of capture was measured and multiplied by the annual recharge rate of 1250 gpd/acre for a recharge in the pumping capture area of 19,463 gpd.

Therefore, the total initial pumping rate based on recharge areas as described above was calculated as:

6,250 gpd + 90,000 gpd + 19,463 gpd =
115,713 gpd or about 116,000 gpd

Theis Method

In the second method of calculating initial pumping the 56 wells calculated by the Theis method previously described were multiplied by the pumping rate of 1.5 gpd (also used in the Theis equation) to obtain a pumping rate of 120,960 gpd, or about 121,000 gpd.

2.5.2.2 Post-Capping Flow Rates. The following two methods were used to calculate the range of pumping rates needed to control contaminated groundwater discharge from the landfill after achievement of 85% reduction in discharge from the landfill.

Darcy Water Budget

Using the Darcy Water Budget, the total pumping rate needed to control the release of contaminated groundwater discharged from the landfill was calculated as follows:

Upgradient Recharge + Cap Leakage + 12-Year Interior
Landfill Drainage + Recharge to Pumping Capture Area
Outside of Landfill = Total Flow to Be Pumped

As previously calculated, the upgradient recharge is 6250 gpd, and the recharge to the pumping capture area outside the landfill is 19,463 gpd.

Leakage through the multilayered cap with partial impermeable membrane was calculated using Darcy's flow equation:

$$Q_{CL} = \frac{K_{CL} H A_{CL}}{L}$$

and applying the following parameters:

K_C = permeability of clay cap, 1.968×10^{-7} ft/min
K_L = permeability of membrane liner, 0.00
H = hydraulic lead, 1 ft
L = length of flow path = 1 ft
A_{CL} = area of the cap, 72 acres clay - 16 acres of liner equals 56 acres

to calculate an average Q_{CL} (flow rate of water through the cap) of 5200 gpd.

Finally, using an initial landfill discharge rate of 100,000 gpd and an 85% reduction in hydraulic head in the landfill after 12 years, the annual release of water from storage after 12 years of drainage would be 14,877 gpd.

Therefore, the total groundwater to be pumped after 12 years would be 6250 gpd + 5200 gpd + 14,877 gpd + 19,463 gpd for a total of 45,970 gpd or approximately 46,000 gpd.

<u>Theis Method With an Extended Period of No Recharge</u>

As compared to the original recharge (or pumping) rate for the 56 pumping wells of approximately 116,000 gpd, the equilibrium pumping rate after 12 years (of approximately 46,000 gpd) represents a 60%

reduction in the original recharge rate. This reduction can be converted to an extended period of no recharge of 250 days ([365 days x 0.60] + 30 days).

The Theis equation was then applied under this extended period of no recharge as follows:

$$Q = \frac{4\pi Ts}{W(u)} \quad \text{and} \quad u = \frac{r^2 S}{4\,Tt_e}$$

where:

Q = discharge or pumping rate, gpd
T = transmissivity of saprolite, 1187 gpd/ft
s = drawdown at a radius of 50 ft from the well, 1 ft
r = radius of observation well from pumping well, 50 ft
te = extended no recharge period
S = storativity of saprolite, 0.001
W(u), u = well function parameters

to obtain an average pumping rate of 1400 gpd/well for/a total of 78,400 gpd for 56 wells, or about 78,000 gpd.

2.5.2.3 Pumping Rates Over 30-Yr Life of System. Using the range of initial and subsequent pumping ranges described above, an estimate of pumping rate ranges over the 30-yr life of the well recovery system was developed as follows:

YEARS AFTER START OF PUMPING	PUMPING RATE (gpd)
0	116,000-121,000
5	76,000-103,000
10	51,000- 86,000
12	46,000- 78,000
15	40,000- 69,000
20	35,000- 60,000
25	33,000- 56,000
30	32,000- 54,000

The initial pumping rates and the 12th year equilibrium rates are those described in the preceding discussions. The pumping rates between the first and 12th years reflect the exponential decline in hydraulic head and groundwater discharge under the landfill to the 85% reduction level reached by year 12. From year 12 onward the much more gradual decline in pumping levels is attributable to concomitantly gradual decline in the remaining 15% landfill groundwater discharge and hydraulic head.

2.5.2.4 Impacts of Pumping Rates. Some reviewers have expressed a concern about the rate of drawdown of contaminated groundwater beneath the landfill and the pumping rate and well spacing to achieve this drawdown. Although a somewhat higher rate of drawdown could be achieved at the site, the actual drawdown achievable will be limited by the hydraulic characteristics of the saprolite, which, as suggested in Chapter 6, need to be defined more precisely. If a more rapid drawdown, and therefore higher pumping rate, is used, a larger treatment facility must be constructed to handle the greater initial flows; therefore, the capital costs as well as O&M costs (for the first 10 to 15 years) will increase. Assuming that the perimeter shallow well pumping system is successful in capturing most of the groundwater leaving the site, there may be no compelling reason to more rapidly draw down the remaining groundwater in light of these additional costs.

Although the perimeter shallow well recovery system will control substantially all of the shallow groundwater (i.e., 90% of the total groundwater flow), complete reduction of the groundwater table below the fill will probably not be possible, because estimated fill depths at the center and perimeter of the site are below the pumping levels of the wells. Since the shallow wells will be constructed only to the base of the saprolite, it is not possible to completely dewater the saprolite within the fill area. On the other hand, some reversal of bedrock aquifer flow directly beneath the landfill will result from a reduction of the hydraulic gradient in the shallow aquifer, such that hydraulic reversal in the uppermost intervals of the bedrock aquifer will cause some upward flow of bedrock groundwater into the shallow aquifer where it can be captured by the shallow aquifer recovery system.

Recharge to Trout Brook will continue from areas downstream of the landfill, although stream segments immediately downstream of the landfill will be somewhat dewatered and sustain only stormwater runoff. Potable wells to the north and northwest of the landfill should not be affected by the well pumping since, as described in the RI, groundwater in the deep potable bedrock aquifer does not flow toward these areas.

2.6 GROUNDWATER/LEACHATE TREATMENT AND DISPOSAL

Both on-site and off-site treatment options were initially evaluated in the RI/FS, including on-site pretreatment of leachate for subsequent discharge to a publicly owned treatment works (POTW) and complete on-site treatment of leachate for direct effluent discharge. Because of the remote location of suitable POTWs and extremely high construction and operation and maintenance costs to treat contaminated groundwater off-site, the on-site pretreatment option was not considered to be cost-effective. Therefore, the collected leachate and contaminated groundwater will be treated at a complete on-site treatment facility located at the headwaters of the East Branch of Trout Brook (Figure 1-1; Plate 1).

Treated effluent will be discharged to the continuously flowing portion of Trout Brook below the confluence of the East and West branches. Trout Brook is classified as an FW-2, Category One, nondegradation water by NJDEP. Chapter 5 describes the draft effluent limitations established by NJDEP for a discharge at this location and their implications for groundwater treatment.

A complete, on-site treatment facility is expected to consist of a series of physical, chemical, and biological treatment processes that may be required to meet NJDEP discharge limitations and will include:

- Equalization/storage to reduce wasteload fluctuations
- Chemical precipitation and sedimentation to remove solids and heavy metals
- Removal of organic compounds (as measured by BOD_5) and ammonia with a biological treatment process such as a rotating biological contactor (RBC)
- Carbon adsorption to remove trace organics, preceded by dual media filtration to remove suspended solids
- Sludge holding tank for transportation of sludge to a local POTW for final treatment and disposal

Figure 2-7 presents a schematic process flow diagram of the liquid treatment components of this system. The suitability of these treatment processes and appropriate sludge handling procedures has not yet been defined in detail and will require additional investigation, as described in Chapter 5.

The initial design capacity of the treatment facility is based on the estimated average amount of groundwater flow expected to be collected at the site perimeter, i.e., approximately 119,000 gpd, to which a 20% safety margin has been applied to account for seasonally high flows. This translates into an initial design flow of 140,000 gpd or about 100 gpm. As described in Section 2.3, the impermeable cap, in combination with the shallow pumping wells, is expected to lower the average groundwater table beneath the landfill and pump the collected groundwater/leachate to the treatment facility. Within 10 to 15 years the combined effects of the groundwater will lower the pumping rate by 40-60%. This decline in treatment flows suggests the need and opportunity to incorporate packaged units that can be taken off-line as treatment flows decline. The design details and timing for such packaged units need to be developed as part of the final design. Because of the uncertainties regarding actual groundwater pumping rates, the operations and maintenance costs (provided in Chapter 7) have been conservatively estimated based on four pumping rate intervals: 0-5 yr at 100 gpd, 6-10 yr at 55 gpd, 11-15 yr at 45 gpd, and 16-30 yr at 35 gpd. These assumptions will overestimate the O&M costs for the first five years.

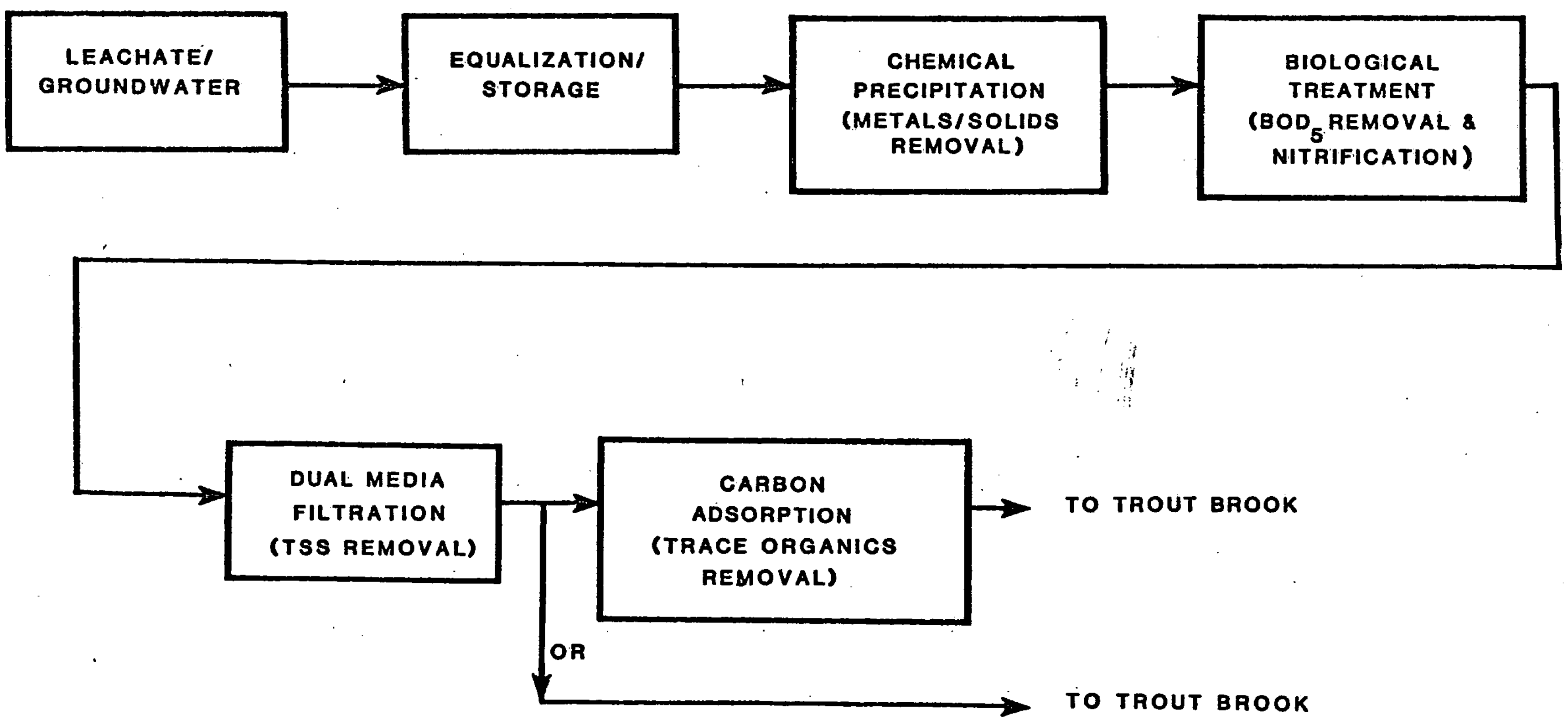
Approximately 1800 gal/day of chemical and biological sludge, at a solids concentration of 2%, would be produced by the treatment processes shown in Figure 2-7. Final treatment/disposal of the sludge should be determined based on chemical characterization and testing of the sludge in a treatability study. The Parsippany-Troy Hills Wastewater Treatment Plant is a possible disposal location since it currently has excess sludge treatment/disposal capacity and will accept a sludge with a solids content of up to 8%. Sludge from the on-site treatment facility at the Combe Fill South Landfill could be transported by tank truck.

2.7 ALTERNATE WATER SUPPLY

The alternate water supply feasibility study (LMS 1986b) recommended that public water be provided, at a minimum, to the residents of Schoolhouse Lane and Parker Lane from the vicinity of Trout Brook to its junction with Schoolhouse Lane. This minimum service area includes approximately 46 homes in the vicinity of the

FIGURE 2-7

PROCESS FLOW DIAGRAM
LEACHATE / GROUNDWATER TREATMENT FACILITIES
COMBE FILL SOUTH LANDFILL



landfill. The water supply study further recommended the extension of potable public water service with a 10-in. diameter main along Parker Road that would allow for fire protection and additional expansion of the system to Route 24 and State Park Road if necessary. However, the additional costs of piping reserves for fire protection or reserves for future needs will not be fundable by Superfund or state money.

The extension of the existing WTMUA water supply system to serve the Parker Road and Schoolhouse Lane areas impacted by the Combe Fill South Landfill appears to be the most practical and feasible source of potable water for the impacted areas. At present, the WTMUA water supply and transmission system is adequate to extend service to the Parker Road and Schoolhouse Lane areas impacted by the Combe Fill South Landfill. While the WTMUA water supply system has some water supply and transmission limitations at present, the system has a sufficiently large customer base to support a reasonable program of improvements to eliminate the deficiencies. Chapter 6 discusses some of the institutional and administrative questions that need to be addressed prior to final design of this water supply extension.

CHAPTER 3

OPERATION AND MAINTENANCE

The following discussion of the operation and maintenance (O&M) needs of the remedial action alternative assumes a minimum project life of 30 years. However, it is possible that some components or subcomponents of the remedial action need to function for only part of the project life. For example, although the treatment facility is originally designed to handle groundwater/leachate flows of 100 gpm, these flows are expected to decrease with time such that between 60 to 80% of the original plant capacity will not be used. Such a situation would ideally be taken care of by using modular treatment units that can be taken out of service when no longer needed.

Conversely, it is also likely that several components, particularly the cap and perhaps the groundwater pumping system, will need to function beyond the 30-yr project life. Unless technical advances create opportunities in the future for implementing other permanent solutions to the problems at the Combe Fill South Landfill, several remedial components may require substantial renovation or reconstruction, neither of which is addressed in this assessment.

3.1 ACCESS ROADS AND SITE SECURITY

The fence, locking gates, and warning signs around the site perimeter will require regular inspection for signs of vandalism or natural wear and tear. Fence sections and the gate itself may require periodic repair and/or replacement. Paved access road segments will require periodic resurfacing and repair of potholes and cracks. Unpaved access roads will have to have gravel replaced and surfaces regraded.

3.2 CAP, TERRACES, AND SURFACE WATER CONTROLS

The primary objective for O&M of the cap and its terraces is to maintain the integrity of the clay layer and the impermeable membrane (where it is present) so as to minimize infiltration. Regular inspection of the cap will reveal cracks, rifts, or sinkholes created by differential settling of waste beneath the cap or unsuccessfully controlled erosion. These cracks, rifts, and sinkholes must be repaired regularly. Shifting sections of the gabion terraces will also require regular repair to help maintain control of surface water runoff. Drainage chutes, berms, and ditches must be kept clear of obstructions and repaired to ensure adequate surface water runoff control. The vegetative cover of the cap must be mowed and reseeded to maintain cap surface integrity. Stray shrub and tree seedlings must be removed to prevent cracking of the cap surface and subsequent increased infiltration. Areas of the cap that begin to show ponding of water or erosion will have to be recontoured and reseeded.

3.3 GAS COLLECTION AND TREATMENT

Because the gas venting and treatment system is forced rather than passive, it will require more manpower and services (e.g., electricity) for proper operation and maintenance. Inspection of the gas vents, piping system, blowers, burners, and associated electrical system will have to be conducted regularly. Piping and vents must be checked for cracks, corrosion due to landfill gases and wastes, and obstructions, and will require replacement or repair as necessary. Blowers and burners need to be inspected regularly for proper operation and will require periodic repair, although they should not need to be replaced during the initial 30-yr system life.

Additional treatment components or a gas reuse system will require correspondingly more elaborate O&M procedures, although the benefits of a methane reuse system may equal or outweigh the additional O&M costs. (As discussed in Chapter 6, methane reuse requires additional analysis prior to final remedial design.)

3.4 SHALLOW GROUNDWATER WELL PUMPING

The entire shallow groundwater (i.e., saprolite) well pumping system of wells, pumps, well pump controls, and collection and utility trenches will require regular inspection to maintain the desired groundwater levels and flow at the site. Because greater volumes of groundwater will be pumped in the first 15 years of the project life, the O&M requirements of the pumping system will likewise be greater during that time, e.g., well pumps and level controls will need more frequent repair/replacement in the early years of the remedial action. Well screens and casings must be inspected for deterioration caused by contact with the waste on-site. Although all wells were assumed to be functional for the entire 30-yr project life, new pumping wells may have to be installed. The leachate/groundwater collection pipelines and utility trenches will also require regular inspection and repair. Inspections and repairs will be made through access manholes in the collection/utility trench. The PVC collection pipeline will require inspection for corrosion or the accumulation of iron bacteria; flushing and chemical treatment may be necessary for the pipeline, or actual segments of the pipe may require replacement.

3.5 GROUNDWATER/LEACHATE TREATMENT AND DISPOSAL

Any on-site groundwater/leachate treatment system will require manpower, chemicals, and electricity to meet the specific needs of the selected treatment components. A complete on-site treatment system will probably require one full-time and one part-time operator. In addition to the operation of the process components and maintenance of the supporting electrical and mechanical equipment, other operation and maintenance needs will have to be provided, such as maintenance of the gas treatment or reuse system and transportation of chemical/biological sludges to an appropriate facility for final treatment and disposal. If packaged process/operation components are used during the initial construction of the treatment system, they can be taken off-line as the groundwater/leachate flows decrease, thus reducing both the initial construction costs and future operation and maintenance needs.

3.6 MONITORING WELLS

In addition to the long-term quarterly monitoring activities described in Chapter 4, the groundwater wells associated with this monitoring program will require periodic inspection to ensure their continued integrity and suitability for use, i.e., well casings and screens may deteriorate, locking caps may be broken, or access to the well may become restricted.

CHAPTER 4

LONG-TERM MONITORING

The long-term environmental monitoring program proposed for the Combe Fill South landfill site includes quarterly sampling and analysis of air, surface water, and groundwater. Particular emphasis is placed on groundwater monitoring because of the need for timely reassessment of the effectiveness of remediation of contamination in the saprolite and bedrock aquifers. A conscientiously applied monitoring program will not only continue to help reassess site conditions and contaminant movement, but will also serve as a trigger for particular O&M routines and as an early warning system for additional remedial actions.

4.1 AIR MONITORING

During construction and for the first several years of operation, quarterly sampling of gaseous and particulate ambient air fractions at upwind, on-site, and downwind locations should be undertaken. At a minimum, samples should be analyzed for volatile organics, methane, and heavy metals since these are the present contaminants of concern. The cost estimate prepared in Chapter 5, however, conservatively assumes that the full suite of priority pollutants will be analyzed for 30 years. Appropriate field and trip blanks and duplicates should also be taken along with these site samples. Should the results of the early air sampling and analysis reveal no or little air migration of contaminants, the scope of the air monitoring program could be limited to sampling the minimum number of fractions once per year.

4.2 SURFACE WATER MONITORING

Surface waters and their associated sediments should be sampled at several locations, including a surface water outlet (e.g., the circumferential drainage ditch outlet to Trout Brook), directly below the treated effluent discharge point in Trout Brook, and at the confluence of Trout Brook and the Black River. Because the headwaters of Trout Brook will consist primarily of surface water runoff from the remediated landfill, another background surface water location should be sampled, e.g., the Black River upstream of its confluence with Trout Brook. All surface water and sediment sites should be sampled quarterly and analyzed for all priority pollutants. Appropriate field and trip blanks and duplicates should also be taken for each quarterly sampling event.

4.3 GROUNDWATER MONITORING

Groundwater monitoring in compliance with RCRA requires, at a minimum, quarterly sampling and analysis of one upgradient and three downgradient wells. The groundwater monitoring program described in the following paragraphs and shown in Figure 4-1 is based on the assumption that public potable water will be supplied to the minimum service area outlined in the water supply feasibility study (LMS 1986b). As such, this groundwater monitoring program is designed to provide detailed and timely reassessment of the location of groundwater contamination and any changes in groundwater movement (in both the saprolite and bedrock aquifers) so that remedial actions, specifically the amount, location, and rate of well pumping, can be adjusted if necessary. However, since the actual area to be serviced with public water may be considerably expanded, as compared to the minimum service area, a smaller scale monitoring program may be appropriate, and therefore this proposed monitoring program should be reevaluated as part of the design phase of this project.

The proposed groundwater monitoring program for the Combe Fill South landfill is a circumferential one as shown in Figure 4-1 and consists of:

- Twenty deep bedrock wells. Nine of these deep wells were constructed as part of the RI. Of the 20 deep wells, two are upgradient. Five are on-site and 13 are downgradient. Several of the deep downgradient wells are coincidentally located with currently existing private potable wells so that these potable wells could be used, obviating the need to install entirely new wells. However, since private potable wells are not constructed to NJDEP monitoring well specifications, the cost estimate provided in Chapter 7 assumes that new wells will be installed.

- Fourteen shallow saprolite monitoring wells. Six of these wells are the same as those installed and sampled during the RI. Of the 14 shallow wells, one is upgradient, four are on-site, and nine are downgradient.

Construction of new monitoring wells and subsequent sampling should follow current NJDEP guidelines to assure accurate, precise, and representative data. Because many of the monitoring wells will be located off-site, all wells should be securely locked for safety, to prevent vandalism, and to assure the integrity of water samples taken from the wells.

If public water is initially provided to a more extensive service area than that stipulated as the minimum service area in the alternate water supply feasibility study (LMS 1986b), then the number of deep monitoring wells can be reduced appropriately. For this technical and cost analysis, however, a maximum number of monitoring wells has been assumed. In addition, although this assessment (and cost estimate in Chapter 5) assumes that each sample will be analyzed for the full suite of priority pollutants, a limited number of analyses could be undertaken and still be effective as a gauge of contaminant migration. For example, groundwater analyses for this site could be limited to volatile organics, base/neutral extractable organics, and heavy metals since these were the contaminants of concern migrating from the landfill in the groundwater.

CHAPTER 5

GROUNDWATER TREATABILITY STUDY

5.1 INTRODUCTION

As described in Chapter 2, complete on-site treatment and discharge to nearby Trout Brook was considered the only viable option and was recommended as part of the proposed remedial action for the site. Trout Brook in the vicinity of the Combe Fill South landfill site is classified as FW-2, Category One, nondegradation water, by NJDEP. Draft effluent limitations for discharge to Trout Brook, promulgated by NJDEP, are shown in Table 5-1. These limitations are stringent and well beyond the limits that conventional secondary treatment processes can achieve. They are achievable, however, by other available but more expensive treatment technologies. The strictness of the proposed effluent limitations and the need to use some unconventional treatment processes require the implementation of a treatability study.

5.2 PROPOSED TREATABILITY STUDY

A work scope for a proposed groundwater/leachate treatability study was prepared by LMS and submitted to NJDEP in August 1986. The objectives of this proposed treatability study were to:

- Determine the best available treatment technologies to treat waste streams, principally landfill leachate and groundwater.
- Produce effluent quality in compliance with NJDEP effluent limitations.
- Develop design criteria and process requirements for full-scale design.

TABLE 5-1

NJDEP DRAFT EFFLUENT LIMITATIONS AS COMPARED TO EXPECTED INFLUENT CHARACTERISTICS

Combe Fill South Landfill

COMPONENT	EFFLUENT LIMITATIONS	EXPECTED AVERAGE INFLUENT CHARACTERISTICS
Conventional Parameters		
Biochemical oxygen demand, 5 day (BOD_5)	8.0 mg/l monthly average 12.0 mg/l weekly average 20.0 mg/l daily maximum 90% removal efficiency	100 mg/l
Total suspended solids (TSS)	8.0 mg/l monthly average 12.0 mg/l weekly average 20.0 mg/l daily maximum 85% removal efficiency	480 mg/l
Total organic carbon (TOC)	10.0 mg/l monthly average 20.0 mg/l daily maximum	510 mg/l
pH	6.5 - 8.5	7.0
Dissolved oxygen (DO)	7.0 mg/l at any time	-
Ammonia, as nitrogen (NH_3-N)	1.0 mg/l monthly average[a]	50 mg/l
Bioassay	No measurable acute toxicity	-
	96-hr LC_{50} < 10% mortality in all samples, including 100% treatment effluent	-
Ames Test	(No numerical limit for mutagenicity)	-
Priority Pollutants		
Volatile and semivolatile organics (NJDEP "toxic" organics)	ND or <5 ppb, for any single compound, daily maximum	300 ppb
Polychlorinated biphenyls (PCBs)	ND or <0.1 ppb, daily maximum	ND
Pesticides	ND or <1.0 ppb, daily maximum	ND
Heavy metals	ND or <50 ppb, total for all metals, daily maximum	710 ppb
Total phenolics	ND or <50 ppb, daily maximum	210 ppb
Total cyanide	ND or <20 ppb, daily maximum	24 ppb

[a]Possible allowances for seasonal variations not quantified.

ND = not detectable.

The following factors relating to the expected influent characteristics and preliminary effluent limitations unique to the Combe Fill South landfill site were the primary factors used in the development of the treatability study work scope:

- The limitations for effluent metals concentration are very stringent.
- Limits may require unusual sulfide precipitation.
- Very low effluent nitrogen limits must be achieved from an influent high in nitrogen.
- There is a possible need for carbon adsorption to achieve low BOD_5 and priority pollutant organic concentration limits in the effluent.
- Stringent effluent limits will result in higher concentrations of contaminants in sludges (particularly chemical sludges), thus requiring further analysis to determine ultimate disposal method.
- High influent total dissolved solids (TDS) may result in bioassay toxicity unrelated to priority pollutants and may have to be treated via such processes as ion exchange.

No decisions have been made by NJDEP as to the final work scope or start-up of the proposed treatability study. Implementation of the treatability study is advisable prior to start-up of final design so that results from the study can be incorporated into the design.

CHAPTER 6

DESIGN AND IMPLEMENTATION PROBLEMS, DATA NEEDS, AND PERMIT REQUIREMENTS

6.1 SPECIAL TECHNICAL PROBLEMS AND ADDITIONAL DATA NEEDS

Prior to or as part of the final design of the recommended remedial actions, a number of technical and administrative questions need to be addressed. The following paragraphs describe some of these outstanding issues.

6.1.1 Grading, Capping, Terracing, and Surface Water Controls

The location, amount, and suitability of local materials for construction of the cap, including local borrow and clay, will have to be detailed. The method of placement and degree of compaction of the clay needed to achieve the desired impermeability must also be assessed. The shear strength of the cap materials must be evaluated in light of grading and capping slope limitations.

Detailed estimates of surface water runoff and infiltration (including an estimate of the production of leachate) should be evaluated as part of the final cap design by using one of several available computer models, such as HELP. This analysis will better define the hydraulic requirements of the shallow groundwater pumping, collection and treatment system, the surface runoff collection systems, and the cost-effectiveness of the partial impermeable membrane in the western portion of the remediated site.

The rate of, and impacts from, differential settling of the cap layers caused by the natural decomposition of underlying wastes will require further analysis. Differential settling may crack the clay layer (thereby increasing infiltration), damage or break the gas collection and transport pipelines, distort surface gradients, or cause slippage of gabion terraces (resulting in ponding of water or cap erosion). Phased construction of the cap layers, allowing settling to occur between application of the layers, may be one way to reduce adverse settling impacts.

The applicability of using such cap materials as filter cloths and an impermeable membrane for remedial action in this landfill environment should be further evaluated by interviewing manufacturers, reviewing recent research applications of such materials, and summarizing the historical use and life of such materials at similar landfill sites.

Additional detail on the short- and long-term impacts on the wetlands and brooks bordering the landfill resulting from cap construction, long-term groundwater pumping, and altered surface runoff patterns needs to be provided. Direct short-term construction impacts to the wetlands can be minimized by judicious implementation of the grading and terracing as proposed. However, long-term impacts from reduced infiltration, lowered groundwater table, and altered surface water runoff patterns may not be as easily mitigated.

Final design of the surface water control measures should be based on additional evaluation of cap-altered runoff patterns by using mathematical models such as HELP, which calculates surface runoff as well as infiltration. Ditches, berms, drainage chutes, and discharge outlets to surface waters can then be designed to control specific storm flows. The need for, and design requirements of, on-site runoff detention basins should also be reevaluated as part of this work.

6.1.2 Active Gas Collection and Treatment

Few conclusive air quality data have been collected on the Combe Fill South Landfill to date. A detailed air sampling program should be carried out specifically to quantify methane and volatile organics being released from the landfill. Ideally, such a sampling program would be conducted by installing gas extraction wells in and around the site and sampling landfill gases at these wells. Based on this sampling program the suitability of flaring, with or without other treatment mechanisms, for removal of methane and some organics should be reevaluated. This information will also be necessary to further evaluate the use of methane as an energy source in the remedial action.

Gas pipe vent size, vent spacing, and blower capacities should also be reevaluated based on revised estimates of landfill gas generation. The compatability of PVC piping and vents to the chemically harsh landfill waste environment will also need to be addressed. Breaking, cracking, and shifting of gas vents and piping as the result of differential settling also need to be evaluated. Finally, the necessity for and specifications of gas venting for health and safety during construction need to be further addressed.

6.1.3 Shallow Groundwater/Leachate Pumping, Collection, Treatment, and Discharge

Before final design of the shallow groundwater/leachate pumping system, the site's groundwater flow patterns and rate and direction of contaminant movement should be made. Although the saprolite was considered to be anisotropic (LMS 1986a), further evaluation of the ramifications of this conclusion on the design, installation, and operation of the pumping well system can be made prior to final design. To accomplish this, another round of sampling of the deep and shallow groundwater monitoring wells constructed during the RI should be conducted in order to gauge any changes in groundwater quality since the sampling done in 1985. During this resampling, static water level measurements should also be taken in each monitoring well. These data should be compared to those in the RI to see whether the conclusions reached regarding groundwater flow, contaminant levels, rate, and direction of movement are still appropriate. In conjunction with this work, full-scale pumping tests should be conducted to help refine the estimates of groundwater flow, rate, and directions used in the calculations for well numbers, well spacing, and drawdown prior to final design.

The need for deep groundwater pumping needs to be assessed in further detail. This evaluation may need to be ongoing (starting in the design phase, continuing through start-up of the remediation, and into the first few years of operation) to accurately assess the needs for deep pumping. However, a long-term pumping test like the one discussed above may provide substantial information for making preliminary decisions.

As a part of final design and start-up of the long-term monitoring program, additional shallow and deep monitoring wells should be installed and sampled. These wells should be located around the site perimeter where little previous information is available or where no groundwater flow from the site was assumed (i.e., north of D-1). This additional groundwater sampling should be done in conjunction with semiannual sampling of private potable wells in the vicinity of the site, which should be conducted until the remedial action is in full operation and the monitoring program has begun.

The depths of the shallow groundwater/leachate pumping wells, their spacing, and pumping rates should be reassessed using the additional monitoring data discussed above in conjunction with such mathematical tools as the HELP model discussed previously. The needs and appropriateness of installing and operating the groundwater pumping and treatment systems prior to final site capping should also be evaluated.

Equipment and controls such as the well pumps, well casing and screen, electrical wiring and controls, and PVC piping for transporting the groundwater to the treatment facility will need to be selected and sized based on hydraulic considerations and groundwater/leachate characteristics. Maintenance procedures for keeping the pumps and piping clear of sediments, iron bacteria, and iron oxide will have to be established. Adverse impacts of differential settling will have to be accounted for in the established O&M procedures, particularly for the transport system.

In order to design the final treatment facility, a bench-scale treatability study of the groundwater leachate at the site should be conducted to determine the most cost-effective and technically appropriate treatment methodologies. As part of this treatability study, final effluent limits must be prescribed by NJDEP. The air pollution control and sludge disposal needs of an on-site treatment facility must also be determined and incorporated into the final design. Likewise, initial design flows need to be calculated and the appropriateness of packaged units assessed to accommodate decreasing groundwater flows.

Although not technically located in a wetland area, an on-site treatment facility is located at the headwaters of the East Branch of Trout Brook and its impacts (as well as those of the effluent discharge) should be evaluated. As mentioned in previous paragraphs, the long-term operational impacts to the streams and wetlands bordering the site should be examined in greater detail.

6.1.4 Public Water Supply to Residents

Although little technical information is needed to design and implement a public water supply system for the residents near the Combe Fill South Landfill, the water supply feasibility study (LMS 1986b) outlined a number of administrative issues that required resolution prior to final design and implementation, including:

- Need for additional water supply well fields to accommodate expected service areas for the WTMUA
- Final service area definitions
- Service flow rates, i.e., whether fire flows should be included in the design
- Administration and billing procedures for new service area that crosses municipal lines

Once these issues are resolved, the design and implementation of the public water supply can be fast-tracked for completion prior to the other remedial action components.

6.1.5 Miscellaneous Data Needs

Because the RI was unable to make a final determination as to the source of higher than normal radioactivity readings in and around the site, a more specific radioactivity/radon sampling and analysis program should be conducted.

6.2 PERMIT IDENTIFICATION AND REQUIREMENTS

The recommended remedial action must be reviewed for its compliance with Federal, state, and local requirements and, where applicable, permit applications must be made. The areas of such environmental requirements and permits may include the following:

- Effluent discharges will be regulated by NJPDES regardless of which discharge location is used, assuming that complete on-site treatment is undertaken.
- Site monitoring must be conducted in accordance with RCRA requirements for postclosure of landfills.
- Floodplain construction and stream encroachment permits may be necessary for work associated with such items as grading, site preparation, cap construction, access road construction, and construction of the treatment facility itself.
- The impact to wetlands must be addressed according to Executive Order 11990.
- The short- and long-term air emissions from the landfill surface, gas venting system, and groundwater/leachate treatment system must meet Federal and state requirements.
- Hauling of any hazardous wastes, including sludges from the treatment facility, must meet state and Federal (RCRA) requirements.
- Stormwater discharges may require point source discharge (NJPDES) permits.
- Monitoring well construction must be done in accordance with well construction permits issued by NJDEP.

- County or local environmental regulations regarding landfill closure or hazardous waste must be addressed.

6.3 ACCESS, EASEMENTS, AND RIGHTS-OF-WAY

Temporary access to the landfill during construction-related activities and permanent access to the site treatment facility for monitoring and inspection of the site facilities are required. The present access to the site is inadequate for these purposes and new and expanded access is necessary. This will require the purchase of the several rights-of-way around the site perimeter. Access must also be obtained for any new monitoring wells constructed off-site, and a right-of-way must be obtained for the effluent outfall line to its discharge located at the junction of the East and West branches of Trout Brook.

6.4 HEALTH AND SAFETY REQUIREMENTS

Based on present landfill conditions, most nonintrusive construction activities have been assumed to require the use of Level C personnel protection equipment and procedures. Intrusive work has been assumed to require the use of Level B protection. These general health and safety needs should continue to be reevaluated and may significantly impact construction costs.

Other specific health and safety needs during construction should also be reassessed, including:

- Control of dust on and near the fill
- Control of the emission of methane and volatile organics

- Use of nonsparking equipment to help prevent fires and explosions in the methane-laden atmosphere of the landfill

- Preparations for emergency fire, hazardous waste, or general health and safety incidents involving on-site workers and nearby residents, along with evacuation routes

CHAPTER 7

COST ESTIMATES

Cost estimates for the recommended alternative are presented in Tables 7-1 through 7-4 and are based on 1986 dollars. Table 7-1 provides an estimate of the costs to conduct the special studies described in Chapter 6 and an estimate of the design costs for the recommended alternative. The total design costs, including a design contingency of 15%, are estimated to be $1,294,000.

Table 7-2 summarizes the implementation costs of the recommended alternative, including direct capital or construction costs and indirect costs such as design, engineering services, legal and administrative, and construction contingencies. With a subtotal of $32,491,000 in direct construction costs and $8,072,000 in indirect costs, the total capital costs of the project are estimated at $40,563,000. Site preparation for the cap, the cap itself, and its impermeable partial membrane and gabion terraces are the most costly of the remedial components and account for 78% of the total construction costs of the project. Costs for the alternate water supply were based on the minimum service area defined in the alternate water supply feasibility study (LMS 1986b).

Operation and maintenance costs for the recommended project are summarized in Table 7-3. These O&M costs vary on an annual basis from $929,000 in the first five years of the operation of the facility to $910,000 in years 6 through 10 of the operation, $903,000 in years 11 through 15, and finally to $893,000 in years 16 through 30. The most expensive O&M item is the analytical services (i.e., laboratory analyses) of the samples taken for the environmental monitoring program; these analytical costs account for over 50% of

TABLE 7-1

RECOMMENDED ALTERNATIVE SPECIAL STUDIES AND DESIGN COSTS

Combe Fill South Landfill

		COSTS ($)
1.	STUDIES	
A.	Groundwater treatability studies	100,000
B.	Deep groundwater pumping feasibility study (with additonal field work including drilling and construction of new wells)	150,000
C.	Additional miscellaneous studies for further evaluation of impermeable membrane, landfill gas reuse, surface water detention basins	30,000
	Subtotal Studies	280,000
2.	DESIGN	
A.	Access roads and security fence	30,000
B.	Clearing, grading, site preparation and excavation	100,000
C.	Cap with impermeable membrane, gabion terraces, and surface water controls	400,000
D.	Active gas collection and treatment	170,000
E.	Shallow aquifer pumping and new monitoring wells	125,000
F.	Groundwater treatment and discharge to Trout Brook	210,000
G.	Alternate water supply	90,000
H.	Design contingency allowance @ 15%	169,000
	Subtotal Design	$1,294,000

TABLE 7-2

RECOMMENDED ALTERNATIVE IMPLEMENTATION COSTS

Combe Fill South Landfill

	COSTS ($)
DIRECT CAPITAL COSTS	
1. Security fencing with locking gate	111,000
2. New monitoring wells installed	270,000
3. Access roads	300,000
4. Site preparation	
a. General site grading	1,497,000
b. Cap perimeter clearing and grading	72,000
c. Excavation in power line right-of-way	767,000
5. Capping and gabion terracing	
a. Multilayer clay cap and revegetation	20,507,000
b. Gabion terracing	1,015,000
c. Impermeable membrane	1,608,000
6. Active gas collection and treatment	1,763,000
7. Surface water controls	
a. Berms and reinforced chutes	185,000
b. Perimeter drainage ditch	336,000
8. Shallow aquifer pumping system	1,296,000
9. Wastewater treatment and discharge to Trout Brook	1,364,000
10. Permanent alternate water supply	1,400,000
Subtotal Direct Capital Costs	32,491,000
INDIRECT CAPITAL COSTS	
1. Special studies and design	1,574,000
2. Engineering services during construction @ 7% of direct capital cost	2,274,000
3. Legal and administrative costs @ 3% of direct capital cost	975,000
4. Construction contingencies @ 10% of direct capital cost	3,249,000
Subtotal Indirect Capital Cost	8,072,000
TOTAL CAPITAL COST	40,563,000

TABLE 7-3

RECOMMENDED ALTERNATIVE OPERATION AND MAINTENANCE COSTS

Combe Fill South Landfill

	COSTS/YEAR ($)
1. Fence inspection, repair fence, replace gates	7,000
2. Long-term monitoring	
a. Quarterly sampling of monitoring wells, air and surface waters, and data analysis	19,000
b. Analytical services	468,000
3. Access road maintenance and repair	2,000
4. Cap maintenance and repair	
a. Inspections; runoff and subsidence repair	43,000
b. Vegetation mowing, fertilizing and reseeding	47,000
c. Gabion terrace maintenance and repair	14,000
d. Impermeable membrane repair and replacement	16,000
5. Active gas venting and treatment maintenance and repair (without gas reuse)	67,000
6. Surface water controls maintenance, and repair	6,000
7. Shallow well pumping maintenance, repair and replacement of pumps	151,000
8. Wastewater treatment and disposal operation and maintenance	
a. Years 1-5 @ 100 gpm	89,000
b. Years 6-10 @ 55 gpm	70,000
c. Years 11-15 @ 45 gpm	63,000
d. Years 16-30 @ 35 gpm	53,000
ANNUALIZED O&M	
Years 1 through 5	929,000
Years 6 through 10	910,000
Years 11 through 15	903,000
Years 16 through 30	893,000

TABLE 7-4

RECOMMENDED ALTERNATIVE PRESENT WORTH

Combe Fill South Landfill

CAPITAL COST ($1,000)	YEARS	O&M COST/YEAR ($1,000)	O&M CAPITIZED[a] ($1,000)	TOTAL PRESENT WORTH[a] ($1,000)
40,563	1- 5	929	3,522	49,172
	6-10	910	2,142	
	11-15	903	1,320	
	16-30	893	1,626	

[a]Interest rate of 10% over 30-yr project life.

the annual O&M costs. The next most expensive O&M item is the maintenance and repair of the shallow aquifer pumping system at approximately $151,000 annually.

As discussed in Chapter 4, the cost estimate for the annual analytical services associated with the environmental monitoring program conservatively assumes that every sample will require analysis for the full suite of priority pollutants. If, however, the analyses were kept to the more pertinent fractions for the particular environmental sample, the annual cost of the analytical services could be reduced by 60%, i.e., to about $200,000 annually. This would translate into an average overall annual O&M savings of about 30%. Proportionally additional capital and O&M costs savings would be realized if fewer monitoring wells are needed.

The present worth of the recommended alternative, $49,172,000, was calculated using a 10% interest rate applied over a 30-yr project life to be consistent with such use by EPA at other CERCLA sites.

CHAPTER 8

DESIGN AND IMPLEMENTATION SCHEDULE

A possible design and implementation schedule for the recommended alternative is summarized in Table 8-1. Total project time is estimated at about 57 months. It is assumed that the design and construction of the alternate water supply will be conducted in parallel with the rest of the remedial efforts; however, they could be fast-tracked to provide alternate water within 12 to 18 months of the start of the project.

Phasing of particular construction activities or remedial unit operations, such as the cap (or its components), fence, groundwater pumping wells, and treatment facility, should also be evaluated and detailed during final design. Such phasing may allow for the completion of some remedial components before others. For example, groundwater pumping and treatment can be constructed and under way before the cap is completed, thereby effecting some, although not all, remediation of the groundwater problems.

TABLE 8-1

RECOMMENDED ALTERNATIVE
DESIGN AND IMPLEMENTATION SCHEDULE

Combe Fill South Landfill

ACTIVITIES	TIME FROM START OF PROJECT (TIME MONTHS)
Completion of additional studies	4 Months
Completion of final design, permit applications, bid documents; bid period opened	16 Months
Bid period closed	18 Months
Contractor selected, contracts negotiated, permits approved	20 Months
Access obtained, properties purchased, construction HASP and QAP prepared	21 Months
Construction started[a]	21 Months
Alternate water supply construction completed[b]	27 Months
Remediation construction completed	57 Months

[a]The actual phasing of construction elements will be determined during the final design and may allow for the completion of some remedial components before others.
[b]The alternate water supply design and construction could be fast-tracked separately from the rest of the remedial activities and be completed within 12 to 18 months from time zero.

APPENDIX A

AQUIFER HYDRAULIC CHARACTERISTICS

APPENDIX A

AQUIFER HYDRAULIC CHARACTERISTICS

Aquifer Characteristics

The aquifer system that underlies and surrounds the Combe Fill South landfill consists of fractured granitic bedrock and an overlying layer of soil and saprolite. It is necessary to consider the aquifer as a two-layered system because the hydrologic properties of these two materials are very different.

Fractured Bedrock. In weathered and fractured bedrock aquifers, such as those that occur beneath Combe Fill South, groundwater is stored and transmitted along discontinuities within the rock mass of the aquifer. These discontinuities may include fractures, joints, cleavage planes, foliations, and schistosity partings, which form an interconnected network for groundwater flow. As described later in this chapter, the most prominent discontinuity features (openings), as determined from examination of outcrops of the bedrock on and near the landfill, are partings parallel to the foliation that is oriented N50°E and dipping 80°SE. In addition, joint sets present in the rock mass are oriented N35-43°E, with a vertical dip nearly parallel to the orientation of the foliation partings. Discontinuities with other orientations were observed, but the major planar features tend to be parallel and subparallel to the foliation. Under these conditions groundwater migration is biased in the direction of the predominant discontinuities. Permeability and transmissivity (ability of rock material and the aquifer to transmit water) is the greatest parallel to these planes and lower perpendicular to the same planes. This directional permeability is referred to as anisotropy.

In order to measure aquifer transmissivity, short duration (4-hr) constant-rate pumping tests were conducted in the nine deep (D-series) wells that were completed in the bedrock. Numerous short duration tests were conducted rather than fewer, longer term tests-because the transmissivity of the fractured aquifer was expected to be extremely variable over the site. The degree of this variability can be best measured by a relatively large number of short-term pumping tests that generally measure transmissivity in the vicinity of the well. Data from the pumping tests and the analysis and calculations associated with this data are shown in Appendix P of LMS 1986a. Table A-1 summarizes the calculated transmissivity values for each pumping test, which range from 25 to 2640 gpd/ft.

The slopes of the pumping test time-drawdown curves for each well can be approximated with a straight line within the first 10-30 min of pumping. Shortly thereafter, the slopes of the time-drawdown curves flatten considerably, indicating the influence of recharge. Transmissivity values for each pumping well were calculated from a straight line fitted to the first 30 min of pumping as recorded on the time-drawdown curves (Appendix P LMS 1986a).

After cessation of pumping, water level recovery was generally recorded over a 2-hr time period. Semilogarithmic plots of the residual drawdown (recovery time) vs the function t/t' were also used to calculate aquifer transmissivity. [The function t/t' is the ratio of time since pumping began (t) to time since pumping stopped (t').] Straight lines were fitted to the recovery curves where t' = 1-10 min and were used to calculate transmissivities. The transmissivities calculated from the recovery and pumping tests were then averaged to obtain a best approximation of overall aquifer transmissivity as shown on Table A-1. Although the range of the

TABLE A-1

SUMMARY OF VALUES CALCULATED FROM PUMPING TESTS

Combe Fill South Landfill

WELL	AQUIFER MATERIAL	AVERAGE TRANSMISSIVITY (T)	T VALUE FROM PUMPING TEST	T VALUE FROM RECOVERY TEST
D-1	Granite	25.2	28.2	22.2
D-2	Granite	254	309.5	199.5
D-3	Granite	70.8	81.2	60.3
D-4	Granite	40.9	46.5	35.2
D-5	Granite	54.7	59.7	49.7
D-6	Granite	66.0	70.8	61.1
D-7	Granite	204	211	198
D-8	Granite	2640	2640	-
D-9	Granite	154	166.1	142
Geometric average		121		

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transmissivities is quite large, other characteristics of the pumping tests are quite similar:

- After a pumping period ranging from as few as 10 to as many as 100 min, the slopes of the time-drawdown curves from the pumping wells in the bedrock aquifer decrease significantly. This reduction probably results from the influence of delayed gravity drainage (or vertical leakage) of water from the overlying saturated saprolite.

- Except at well D-1, the slope of the recovery curve was always steeper than the slope of the drawdown (pumping curve). As a result, transmissivity values calculated from the recovery curves were lower than those calculated from the drawdown curves. This indicates that the aquifer has undergone a reduction in storage (storativity), probably due to consolidation of the saprolite aquifer or, more likely, entrapment of air within the dewatered portion of the aquifer.

- Six of the pumping wells (wells D-1, D-4, D-5, D-7, D-8, and D-9) were located in close proximity to shallow observation wells constructed in overlying saturated soil and saprolite. (Wells D-1, D-3, and D-6 had no accompanying observation wells; both drawdown and recovery were measured in the same well.) However, drawdown in the observation wells occurred only during four of the pumping well tests (D-1, D-4, D-7, and D-9). In all cases the slopes of the time-drawdown curves for the observation (recovery) wells were much lower than the slopes of the time-drawdown curves for the pumping wells because the observation wells were screened in the saprolite while the pumping wells were tapping the bedrock. Thus, the calculated higher transmissivities in the observation wells reflect the saprolite, not the bedrock. These lower transmissitivities in the bedrock wells may also be related to the influence of frictional well losses on drawdown values in the pumping well or to the effects of time lag between pumping in one well and drawdown response in an observation well.

Because no observation wells were available in the bedrock aquifer within the influence of the bedrock wells that were test pumped, no measure of directional transmissivities was made. Such measurements would have been useful in quantitatively characterizing the anisotrophy of the bedrock aquifer. For the same reason, the storativity of the bedrock aquifer was not measured.

Saprolite Aquifer. The unconsolidated overburden, predominantly saprolitic in nature and including landfilled wastes in the filled areas, is generally saturated across the site. As such, this unit, termed the saprolite aquifer, constitutes a significant aquifer on the site. It has a saturated thickness ranging from 0-40 ft deep with an average thickness of 30 ft as shown on Plate 6 of LMS 1986a. The maximum saturated thickness occurs at well D-6, one of the highest elevations on the landfill, and consists almost entirely of saturated wastes. Generally, the saturated waste thicknesses are 30-35 ft as shown on Plate 6 of LMS 1986a. Substantial thicknesses of saturated saprolite occur along the northern perimeter of the landfill between wells D-4 and D-1; along the northeast perimeter between wells D-5 and DW-4; along the entire southeast perimeter, parallel to the NJP&L power line; and along the southwest perimeter, from well D-7 to well D-9. As such, groundwater and leachate flows away from the landfill within the saprolite aquifer.

The saprolite consists of sandy silt to gravelly silt, and is substantially more porous than the bedrock aquifer because of its unconsolidated nature. For this reason, permeability measurements and transmissivity calculations for the saprolite aquifer were also made from data obtained in slug and pump tests of wells screened in the saprolite. On 17, 18, and 19 April 1985 slug tests were conducted on wells S-1, S-2, S-4, S-5, and S-6. These wells consist of 4-in. diameter stainless steel casings with a 10-ft section of 20-slot screen. The screens were placed in saprolite intervals. The slug tests used a 0.193-ft diameter by 6 ft solid stainless steel slug to alternately raise and lower the well's static water level. The change in water level over time was recorded in each well by use of a pressure transducer connected to a strip chart recorder.

To perform a slug test, a slug was lowered into the test well and then quickly and smoothly submersed below the original static water level, creating an instantaneous water level recovery. After the water level declined to the static condition, the slug was withdrawn and water in the well was allowed to recover to its original height. The rate at which the water level declines or recovers during these tests is a direct measure of the permeability or hydraulic conductivity of the saprolite aquifer. A cycle of one insertion and one withdrawal constitutes two permeability tests. At least four permeability tests were conducted on each well.

The slug test results were analyzed by use of the method developed by Bouwer and Rice (1976). Individual analyses for each well tested are presented in Appendix Q of LMS 1986a. The calculated permeabilities are shown on Table A-2 and range from 10.48 to 373.8 gpd/ft^2, with a geometric average permeability of 47.6 gpd/ft^2.

Based upon the saturated thicknesses of the saprolite aquifer at each well (from Plate 6 in LMS 1986a), transmissivities for wells S-1, S-2, S-4, S-5, and S-6 were calculated. These values are summarized in Table A-2 and range from 314 to 7100 gpd/ft. The geometric average transmissivity for the saprolite aquifer, based upon the slug tests, is 1187 gpd/ft. Compared to the bedrock aquifer (Table A-1), the transmissivity of the saprolite aquifer is an

TABLE A-2

SUMMARY OF PERMEABILITY AND TRANSMISSIVITY VALUES OF SAPROLITE DERIVED FROM SLUG TESTS AND PUMPING TESTS

Combe Fill South Landfill

WELL	PERMEABILITY (gpd/ft^2)	TRANSMISSIVITY (gpd/ft)
S-1	373.8	7100
S-2	14.97	494
S-3[a]	28.9	694
S-4	10.48	314
S-5	288	6050
S-6	14.4	605
Geometric average	43.8	1187

[a]Values for well S-3 derived from well S-3 pumping test. All other values derived from slug tests.

order of magnitude higher (121 gpd/ft for the bedrock as compared to 1187 gpd/ft for the saprolite). Thus, the flow of groundwater and leachate from the saprolite aquifer becomes an important consideration in the overall evaluation of groundwater flow.

A 4-hr constant-rate pumping test was conducted using monitoring well S-3, which is screened in the saprolite, to correlate slug test permeabilities. (The log of well S-3 appears in Appendix E of LMS 1986a.) A transmissivity of 694 gpd/ft calculated from the time-drawdown curve from the pump test of well S-3 is in good agreement with those for the other S-series wells.

Based on these test results and calculations, the following average transmissivity values are assumed to be representative of bedrock and saprolite aquifers.

AQUIFER	TRANSMISSIVITY
Bedrock	121 gpd/ft
Saprolite	1187 gpd/ft

The storativity of the saprolite was not measured in the field during the remedial investigation. To determine storativity requires monitoring of a well in the saprolite adjacent to a pumping well in the saprolite. No such monitoring well was available during the pumping of well S-3.

<u>Water Table Configuration</u>. Water levels were frequently measured in 22 monitoring wells from 29 November 1984 to 28 August 1985 at the Combe Fill South site. These water level measurements are summarized on Table A-3. In addition, water level measurements were made in private wells during the sampling of the potable wells

TABLE A-3

COMBE FILL SOUTH
STATIC WATER LEVELS

Static Water Level Elevation (ft)

WELL NO.	TOC Elev. (ft.)	1984 11/29	1984 12/6	1984 12/13	1984 12/19	1984 12/27	1985 1/3	1985 1/8	1985 1/15	1985 1/17	1985 1/23	1985 1/29	1985 4/22	1985 8/28
SB-1	850.35	--	814.30	814.08	813.85	814.20	815.91	816.30	816.05	815.75	--	815.89	815.00	813.52
SB-2	812.76	792.36	792.72	792.84	792.86	791.88	793.28	792.51	792.41	792.38	--	793.38	793.59	793.11
SB-3	815.01	793.01	793.54	794.66	793.49	793.97	794.43	794.56	794.25	794.19	--	793.59	792.43	792.68
SB-4	794.15	788.45	788.98	789.47	788.71	788.84	789.41	789.36	789.32	789.35	789.05	789.27	788.84	789.19
D-1	837.72*	812.87	812.82	815.62	812.84	813.32	812.72	812.77	812.62	812.56	812.67	812.49	812.85	810.59
D-2	794.47	--	--	--	--	--	--	--	--	--	--	--	787.97	--
D-3	826.09	--	--	--	--	--	--	--	778.59	778.46	--	779.13	779.78	778.07
D-4	803.69	--	--	--	--	--	--	--	--	--	795.29	795.69	796.03	794.67
D-5	843.50	808.50	808.40	807.50	807.81	807.87	807.48	807.55	807.50	807.38	--	807.42	807.42	806.27
D-6	872.32	--	--	--	--	--	--	--	--	--	--	809.74	808.81	808.26
D-7	792.65	--	--	--	787.44	787.73	787.94	787.80	787.65	787.33	787.31	786.88	787.24	787.15
D-8	810.16	797.41	798.96	798.70	798.62	798.74	799.15	799.03	798.97	798.96	798.76	798.47	798.10	797.03
D-9	809.24	--	--	--	--	782.97	783.50	783.68	783.42	783.31	--	783.03	781.95	781.59
S-1	793.67	--	--	--	788.75	788.96	789.12	789.09	788.97	788.92	788.60	787.96	787.59	788.25
S-2	817.92	--	--	--	--	--	--	799.32	799.05	799.00	--	19.13	19.76	21.42
S-3	809.93	--	--	--	--	--	786.26	786.14	786.01	786.01	--	785.35	784.45	784.26
S-4	810.33	--	799.08	798.83	798.75	798.89	799.31	799.15	799.08	799.06	798.33	798.00	797.66	796.58
S-5	804.77	--	--	--	--	--	--	--	--	--	795.07	796.50	796.84	795.33
S-6	840.09	--	--	--	--	--	--	813.34	813.19	813.11	813.44	813.19	813.38	811.49
SW-2	799.08	795.58	796.00	795.33	795.41	795.48	795.57	795.20	795.33	795.46	--	793.91	--	792.20
SW-4	785.31	783.31	783.31	783.31	783.31	783.31	783.31	783.31	783.31	783.31	--	--	--	783.39
DW-4	820.87	--	--	--	--	--	--	--	--	--	--	--	--	797.60

* Bottom of box

where accessible. These water level measurements were used in conjunction with stream position, topography, and geology to develop the regional and local (on-site) water table contour maps included in Figure 4-9 and Plate 7 of LMS 1986a, respectively.

These illustrations indicate that the water table configuration is a subdued version of surface topography. A major groundwater divide runs through the landfill in a northeasterly direction (see Figure 4-9 of LMS 1986a) and directs flow northwest to Tanners Brook, southwest to the West Branch of Trout Brook, northeast to the unnamed tributary of the Black River, and southeast to the East Branch of Trout Brook. As shown on Plate 7 of LMS 1986a, the horizontal hydraulic gradient of the water table is generally 0.01-0.03 ft/ft.

The water table contour map (Plate 7 of LMS 1986a) is a best fit of the water level measurements taken from all wells on 28 August 1985 (also shown on Plate 7 of LMS 1986a). Differences in water levels between the saprolite and bedrock aquifers are described in the following section.

On the landfill, the depth to the water table ranges from 5 ft near wells S-1 and D-7 at the southeast corner of the fill to 65 ft under the northernmost portion of the site between wells D-5 and D-1. Seasonal fluctuations in water levels over the nine-month period of water table measurements were no greater than 3-5 ft. However, because the monitoring of water levels was not continuous and some of the wells (wells D-2, D-3, D-4, and D-6) were not monitored through the entire period, it is possible that water level fluctuations may occur over a greater range. Water levels in wells such as D-6, located in the higher portions of the groundwater flow system, may experience water level fluctuations of 15 ft or more.

Groundwater Discharge as Calculated by Darcy's Law. The groundwater discharge from the landfill was also computed using a form of Darcy's Law. The same flow paths, shown on Plate 7 of LMS 1986a and used in the previous USGS baseflow method, were used for this calculation; however, here they can be separated into saprolite and bedrock components within each flow path. Using the transmissivities measured during the pumping and slug tests, and applying the assumed 2.5:1 anisotrophic permeability ratio discussed previously, a set of aquifer parameters specific to saprolite and bedrock were selected for each flow path. The quantity of groundwater flow in each path was estimated by use of the following equation, a modified version of Darcy's Law:

$$Q = TiW$$

where

Q = quantity of groundwater flow in gallons per day (gpd)
T = transmissivity of aquifer in gpd/ft
i = hydraulic gradient of water table
W = width of flow path in ft

This equation can also be used by replacing the transmissivity (T) with the hydraulic conductivity or permeability (k) such that:

$$T = kd$$

where

k = hydraulic conductivity in gpd/ft^2
d = thickness of the aquifer in ft

The average transmissivities (T) and hydraulic conductivities (k) calculated from the pumping tests and slug tests (121 and 43.8 gpd/ft^2, respectively) were used to calculate the average T_{max} and K_{max} (191.3 and 69.3 gpd/ft^2, respectively) along the preferential direction of permeability (N50°E). From this, the angle between the projected flow direction and the direction of T_{max} or K_{max} was measured so that the actual T and k values in the flow directions could be used for the flow calculations. The calculated T and k values for each flow channel and resultant flow calculations are shown on Table A-4. The total groundwater flow in both aquifer layers in each of the six flow channels is also shown on Plate 7 of LMS 1986a.

In flow path 1, for example, the directional hydraulic conductivity (permeability) of the saprolite was calculated as 68 gpd/ft^2. The average saturated thickness along the scaled width (W) of this flow channel (1275 ft) is 26 ft. The flow channel has a hydraulic gradient of 0.021 (i). Substituting these statistics into the above equation yields a flow rate (Q) in the saprolite/unconsolidated aquifer of approximately 47,338 gpd as follows:

$$Q = KdiW$$
$$Q = (68\ \text{gpd/ft}^2)\ (26\ \text{ft})\ (0.021)\ (1275\ \text{ft})$$
$$Q = 47{,}338\ \text{gpd}$$

For the same flow pathway, the directional transmissivity for the bedrock aquifer was 188 gpd/ft and the groundwater discharge calculated for the bedrock aquifer through the flow channel was 5034 gpd, calculated as follows:

$$Q = Tiw$$
$$Q = (188\ \text{gpd/ft})\ (0.021)\ (1275\ \text{ft})$$
$$Q = 5034\ \text{gpd}$$

The total groundwater flow through this flow path is the combined flow through the saprolite and bedrock portions of the aquifer, i.e., 52,372 gpd. The results of similar calculations for the other flow channels are shown on Table A-4 and indicate a combined

TABLE A-4

GROUNDWATER FLOW CALCULATIONS
COMBE FILL SOUTH LANDFILL

Groundwater Flow Channel	Geologic Material	Transmissivity(T) Along Flow Path (gpd/ft)	Hydraulic Conductivity(k) Along Flow Path (gpd/ft^2)	Aquifer Thickness (d) (ft)	Gradient (i)	Width of (W) Channel (ft)	Groundwater Flow (Q) (gpd)	Net Flow Channel Groundwater Flow (Q) (gpd)
1	Saprolite	----	68	26	0.021	1275	47,338	52,372
	Granite	188	----	---	0.021	1275	5,034	
2	Saprolite	----	65	25	0.025	1120	45,500	50,540
	Granite	180	----	---	0.025	1120	5,040	
3	Saprolite	----	34	29	0.014	490	6,764	7,409
	Granite	94	----	---	0.014	490	645	
4	Saprolite	----	38	14	0.017	850	7,687	9,204
	Granite	105	----	---	0.017	850	1,517	
5	Saprolite	----	43	27	0.005	390	2,264	2,498
	Granite	120	----	---	0.005	390	234	
6	Saprolite	----	68	38	0.006	820	12,713	13,633
	Granite	187	----	---	0.006	820	920	
						Totals: Saprolite	122,266	Total: 135,656
						Granite	13,390	

Notes: $Q = TiW$
$Q = kdiW$

groundwater flow of approximately 135,656 gpd. On average, the saprolite aquifer layer conducts nearly nine times the flow of the granite bedrock.

Groundwater Flow Conclusions. The two values calculated for the total quantity of groundwater flow from the landfill area as follows:

METHOD	GROUNDWATER FLOW RATE (gpd)
USGS streamflow records	110,880
Darcy's Law calculation	135,656